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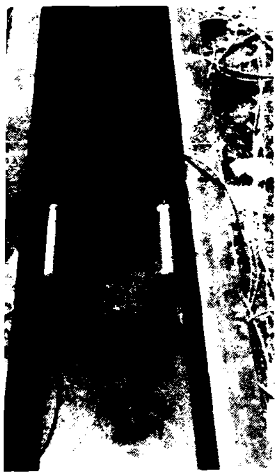

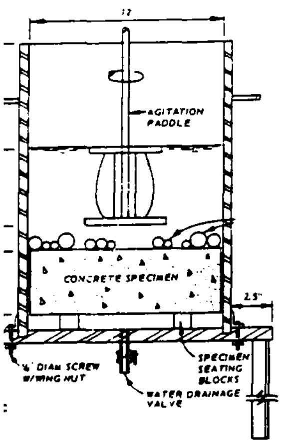

REPAIR, EVALUATION, MAINTENANCE, AND REHABILITATION RESEARCH PROGRAM

TECHNICAL REPORT REMR-CS-32

PROPERTIES OF SILICA-FUME CONCRETE

by

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CS	Concrete and Steel Structures	EM	Electrical and Mechanical
GT	Geotechnical	EI	Environmental Impacts
HY	Hydraulics	OM	Operations Management
CO	Coastal		

COVER PHOTOS:

TOP – Position of strain meters in beam mold.

BOTTON – Abrasion-erosion test aparatus.

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13. ABSTRACT *(Maximum 200 words)*

Two major applications of silica-fume concrete within the Corps of Engineers were to repair abrasion-erosion damage in the stilling basin at Kinzua Dam and in the concrete lining of the low-flow channel, Los Angeles River. In each case, concrete cracking occurred during the repair. Apparently, this cracking has not significantly affected the performance of the concrete in resisting abrasion-erosion damage. However, such cracking could limit the use of silica-fume concrete in other repair and rehabilitation applications.

This study was conducted to determine those properties of silica-fume concrete which might affect cracking and to develop guidance for minimizing cracking problems associated with the use of such concrete in future repair projects. Tests included compressive and tensile splitting strengths, modulus of elasticity, Poisson's ratio, ultimate strain capacity, uniaxial creep, shrinkage, coefficient of thermal expansion, adiabatic temperature rise, and abrasion erosion.

(Continued)

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None of the material properties of silica-fume concrete reported herein, with the possible exception of autogenous shrinkage, indicate that this material should be significantly more susceptible to cracking as a result of restrained contraction than conventional concrete. In fact, some material properties, particularly ultimate tensile strain capacity, would indicate that silica-fume concrete should have a reduced potential for cracking.

Silica fume offers potential for improving many properties of concrete. However, the very high compressive strength and resulting increase in abrasion-erosion resistance are particularly beneficial in repair of hydraulic structures. These concretes should be considered in repair of abrasion-erosion susceptible locations, particularly in those areas where locally available aggregate might not otherwise be acceptable.

The potential for cracking of restrained concrete overlays, with or without silica fume, should be recognized. Any variations in concrete materials, mixture proportions, and construction practices that will minimize shrinkage or reduce concrete temperature differentials should be considered. Where structural considerations permit, a bond breaker at the interface between the replacement and existing concrete is recommended.

PREFACE

The study reported herein was authorized by Headquarters, US Army Corps of Engineers (HQUSACE), under Civil Works Research Work Unit 32303, "Application of New Technology to Maintenance and Minor Repair," for which Mr. James E. McDonald, US Army Engineer Waterways Experiment Station (WES), Structures Laboratory (SL), is Principal Investigator. This work unit is part of the Concrete and Steel Structures Problem Area of the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program sponsored by HQUSACE, for which Mr. McDonald is the Problem Area Leader. The Overview Committee at HQUSACE for the REMR Research Program consists of Mr. James E. Crews (CECW-OM) and Dr. Tony C. Liu (CECW-EG). Technical Monitor for this study was Dr. Liu. Mr. Jesse A. Pfeiffer, Jr. (CERD-C) was the REMR Coordinator at the Directorate of Research and Development, HQUSACE. Program Manager for REMR is Mr. William F. McCleese (CEWES-SC-A).

The study was performed at WES under the general supervision of Mr. Bryant Mather, Chief, SL, and Mr. Kenneth L. Saucier, Chief, Concrete Technology Division (CTD), and under the direct supervision of Mr. McDonald, who prepared this report. Concrete casting and testing activities were monitored by Mr. Donald M. Walley, CTD.

COL Larry B. Fulton, EN, was Commander and Director of WES during publication of this report. Dr. Robert W. Whalin was Technical Director.

CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
cubic yards	0.7645549	cubic metres
degrees (angle)	0.01745329	radians
Fahrenheit degrees	5/9	Celsius degrees or kelvins*
gallons (US liquid)	3.785412	litres
inches	25.4	millimetres
kips (force)	4.448222	kilonewtons
pounds (force)	4.448222	newtons
pounds (force) per square inch	0.006894757	megapascals
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic yard	0.5932764	kilograms per cubic metre
pounds (mass) per square foot	4.882428	kilograms per square metre

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F - 32)$. To obtain Kelvin (K) readings, use: $K = (5/9)(F - 32) + 273.15$.

PROPERTIES OF SILICA-FUME CONCRETE

PART I: INTRODUCTION

Background

1. Laboratory tests have shown that the addition of an appropriate amount of silica fume and a high-range water-reducing admixture (HRWRA) to a concrete mixture will greatly increase compressive strength. This, in turn, increases abrasion-erosion resistance (Holland 1983, 1986a, 1986b). As a result of these tests, concretes containing silica fume were used by the US Army Engineer Districts, Pittsburgh and Los Angeles, to repair abrasion-erosion damage in the stilling basin at Kinzua Dam (Holland et al. 1986) and in the concrete lining of the low-flow channel, Los Angeles River (Holland and Gutschow 1987), respectively. In each case, concrete cracking occurred during repair. At Kinzua Dam, the cracks usually appeared 2 or 3 days after concrete placement. The widths of the cracks at the surface were initially 0.01 to 0.02 in.* and decreased with depth. Ultimately, the cracks were primarily attributed to restraint of volume changes resulting from thermal expansion and contraction and, possibly, autogenous shrinkage. Several different approaches to eliminate or minimize the cracking were attempted; however, no overall solution to the problem with cracking was found. An inspection of the stilling basin by divers approximately 3-1/2 years after the repair indicated that the maximum depth of erosion, located along joints and cracks in the slabs, was approximately 1 in.

2. Apparently, cracks in the concretes containing silica fume have not significantly affected their performance in resisting abrasion-erosion damage. However, such cracking could limit the use of silica-fume concrete in other repair and rehabilitation applications.

Purpose

3. The purpose of this study was to determine those properties of

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* A table of factors for converting non-SI units of measurements to SI (metric) units is presented on page 3.

silica-fume concrete which might affect cracking and to develop guidance to avoid or minimize cracking problems associated with the use of silica-fume concrete in future repair projects.

Scope

4. Concrete materials and mixture proportions similar to those used in the Kinzua Dam repair were obtained for casting laboratory test specimens. Tests included compressive strength, splitting tensile strength, modulus of elasticity, Poisson's ratio, ultimate strain capacity, uniaxial creep, shrinkage, coefficient of thermal expansion, adiabatic temperature rise, and abrasion erosion resistance. Results of these tests were compared with the results of tests on similar concretes without silica fume which were used on recent Corps projects.

PART II: MATERIALS AND TEST SPECIMENS

Materials

5. The aggregates, cement, and silica fume were supplied by the Pittsburgh District. The remaining materials were laboratory stock at the US Army Engineer Waterways Experiment Station (WES). All of the materials used are described in the following paragraphs.

Aggregates

6. The coarse aggregate used, WES, Structures Laboratory (SL) Serial No. CL 55-MG-1, was from Allegheny Minerals Company, Harrisboro, PA. The crushed limestone had an absorption of 0.41.

7. The fine aggregate used, SL Serial No. CL 55-5-1, was from Tionesta Sand and Gravel Company, Tidiout, PA. The washed pit sand had an absorption of 2.17.

Cement

8. The cement used, SL Serial No. CL 55-C-1, was from Armstrong Cement Company, Cabot, PA. The cement met the requirements of American Society for Testing and Materials (ASTM) C 150-86 (ASTM 1987e) for a Type I (low-alkali) cement.

Mineral admixture

9. The silica fume used, SL Serial No. CL 55-AD 847, was from Elkem Chemicals Company, Pittsburgh, PA. Silica fume is finely divided dust collected at electric furnaces where silicon or ferrosilicon is being made. This material fills spaces between cement particles that otherwise would be waterfilled, thus producing stronger concrete. According to the supplier, material characteristics were as follows:

Test	Result
Fineness	274,950 cm^2/gm
SiO_2 content	94.52%
Loss on ignition	2.63%
Moisture content	0.34%

10. The silica fume used at Kinzua Dam and in this study was supplied as slurry in 55-gal drums. According to the supplier, slurry proportions for a 1-cu-yd batch of concrete were as follows:

Material	Weight, lb
Water	134
Silica fume	118
Chemical admixture	11

Chemical admixture

11. The HRWRA used was provided as part of the silica-fume slurry. This admixture permits workable concrete of low water content and, hence, high strength.

Test Specimens

12. A concrete mixture proportioned with 3/4-in. nominal maximum size aggregate for 12,500-psi compressive strength at 28 days was used to cast all test specimens. The water-cement (w/c) plus silica-fume ratio was 0.28. Mixture proportions for a 1-cu-yd batch were as follows:

Material	Weight, lb
Cement, Type I/II	650
Silica-fume slurry	263
Coarse aggregate	1,637
Fine aggregate	1,388
Water	85

Slump and air content of the freshly mixed concrete averaged 9-1/4 in. and 2.2 percent, respectively. Test specimens were fabricated according to applicable provisions of ASTM C 192-81 (ASTM 1987a) as described in the following paragraphs.

Strength and elasticity

13. Twenty-four 6- by 12-in. cylinders were cast for strength and elasticity tests. Because of the limited supply of concrete materials, only two specimens were cast for each test condition. After they were cast, the cylinders were cured in their molds under moist burlap for 24 hr; then the molds were stripped, and the specimens were stored at 73° F in a moist curing room until the time of testing.

Creep and drying shrinkage

14. Ten 6- by 16-in. cylindrical concrete specimens, each containing an embedded Carlson strain meter, were fabricated for creep and shrinkage testing. These specimens were cast horizontally in a steel mold which maintained

parallelism of the 1-in. end plates. After they were consolidated on a vibrating table, the specimens were troweled to complete the circular cross section and then placed under moist burlap. After approximately 24 hr, the specimens were stripped and stored in a moist curing room at 73° F until the time of testing.

Autogenous volume change

15. Four 6- by 16-in. cylindrical specimens, each containing an embedded Carlson strain meter, were fabricated for autogenous volume-change testing. Casting procedures were the same as those previously described for the creep and drying shrinkage specimens. The unrestrained volume-change specimens were demolded 13.25 hr after casting when the concrete reached final set. Following initial strain and temperature measurements, the test specimens were continuously stored in a moist curing room.

16. Six restrained volume-change specimens, prisms 3-in. square with a gage length of 10 in., were fabricated in accordance with ASTM C 878-87 (ASTM 1987h). The specimens were demolded approximately 24 hr after casting. Following initial length measurements, two specimens each were cured under the following conditions: plastic bags in air, moist curing room, and lime-saturated water, all at 73° F.

Ultimate strain capacity

17. Twelve 12- by 12- by 66-in. concrete beams were cast in steel forms for ultimate strain-capacity tests. Strain meters were positioned in the forms parallel to the tensile and compressive faces, 1-1/2 in. from the concrete surface, and centered within the middle one-third of a 60-in. simple span (Figure 1), prior to concrete placement. All beams were consolidated internally, surface finished, and cured in their molds under moist burlap at 73° F for a period of approximately 24 hr. The beams were then stripped, rotated 90 deg, and tested or stored under moist burlap pending testing as appropriate.

Abrasion-erosion

18. Three concrete cylinders, 4 in. high with a 11-3/4-in. diam, were fabricated for abrasion-erosion testing. After consolidation and surface finishing, specimens were cured in their molds under moist burlap for approximately 24 hr. The cylinders were then stripped and stored in a tank of lime-saturated water until time of testing.

Figure 1. Position of strain meters in beam mold

Coefficient of thermal expansion

19. Four 6- by 16-in. cylindrical concrete specimens, each containing an embedded Carlson strain meter, were fabricated for coefficient of thermal expansion testing. Casting and curing procedures were the same as those previously described for the creep and shrinkage specimens.

Adiabatic heat rise

20. A 30- by 30-in. concrete cylinder containing five electrical resistance thermometers was fabricated for the adiabatic temperature-rise test. Immediately after the concrete was placed and consolidated, the cover for the sheet metal specimen container was soldered in place. The specimen was then insulated and placed in the test cabinet.

PART III: RESULTS AND DISCUSSION

Elastic Properties

Compressive strength and splitting tensile strength

21. Compressive strength and splitting tensile strength tests were conducted at 1, 3, 7, 28, 90, 180, and 365 days. Compressive strength testing was done in accordance with ASTM C 39-86 (ASTM 1987d). Splitting tensile strength testing was done in accordance with ASTM C 496-86 (ASTM 1987f). Results are presented in Table 1.

22. Compressive strengths of the silica-fume concrete ranged from 6,080 psi at 1 day to 14,910 psi at 1 year. The concrete gained strength at a very rapid rate with 1- and 3-day compressive strengths of 6,080 and 8,200 psi, respectively (Figure 2). The 28-day compressive strength (14,280 psi) was 96 percent of the strength at 1 year. In comparison, the compressive strength of conventional concrete, containing similar limestone aggregate, without silica fume and HRWRA, was only 5,710 psi at 28 days. The w/c ratio of the conventional concrete was 0.45 (Holland 1983).

23. Splitting tensile strengths ranged from 500 psi at 1 day to a maximum of 1,015 psi at 90 days (Figure 3). Splitting tensile strengths at the various ages ranged from 5.8 to 8.2 percent of the compressive strength at the same age. The higher percentages, 8.2 and 8.0 percent, were at the earlier ages of 1 and 3 days, respectively. Overall, splitting tensile strengths averaged approximately 7 percent of comparable compressive strengths. This value is in general agreement with the approximately 9 percent reported by Saucier (1984) for a range of silica-fume concrete mixtures and the normally accepted value of 10 percent for conventional concrete.

Modulus of elasticity and Poisson's ratio

24. The modulus of elasticity and Poisson's ratio were obtained from cylinders tested for compressive strength. Prior to testing, two electrical resistance wire strain gages were mounted diametrically opposite each other along both the longitudinal and transverse axes of each specimen. An X-Y plotter was used to graphically record stresses and strains. Results of these strain measurements were used to compute the secant modulus of elasticity and

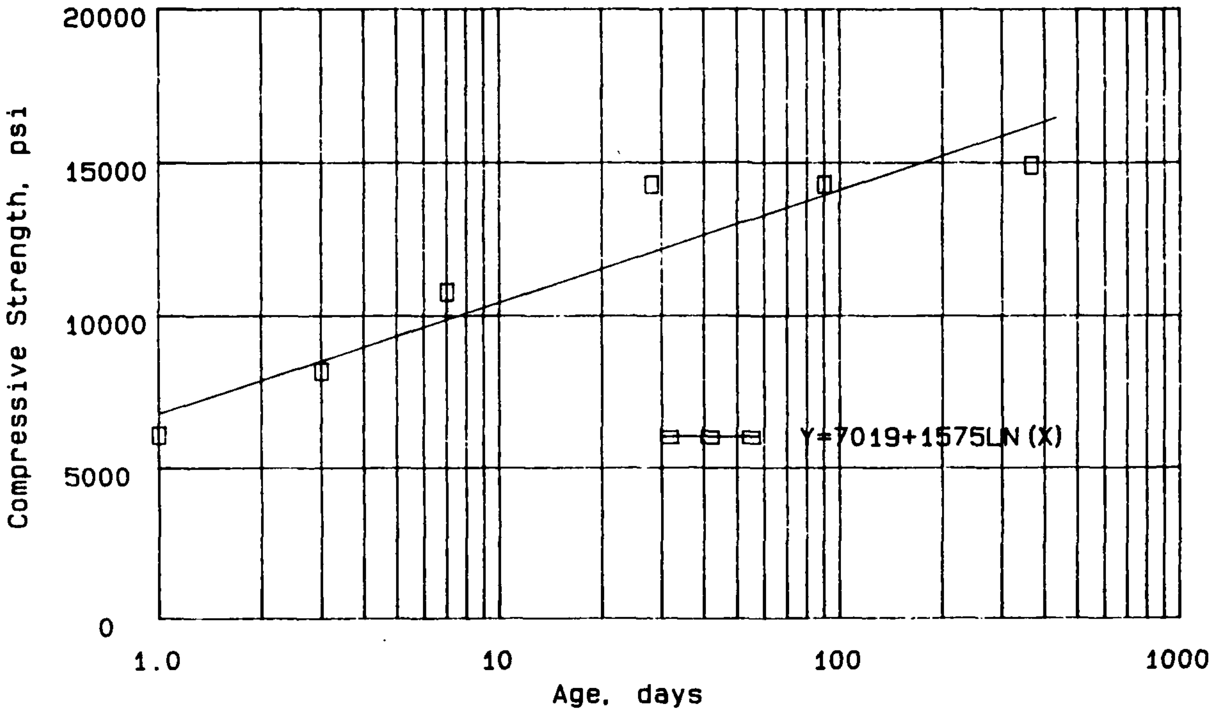

Figure 2. Compressive strength gain

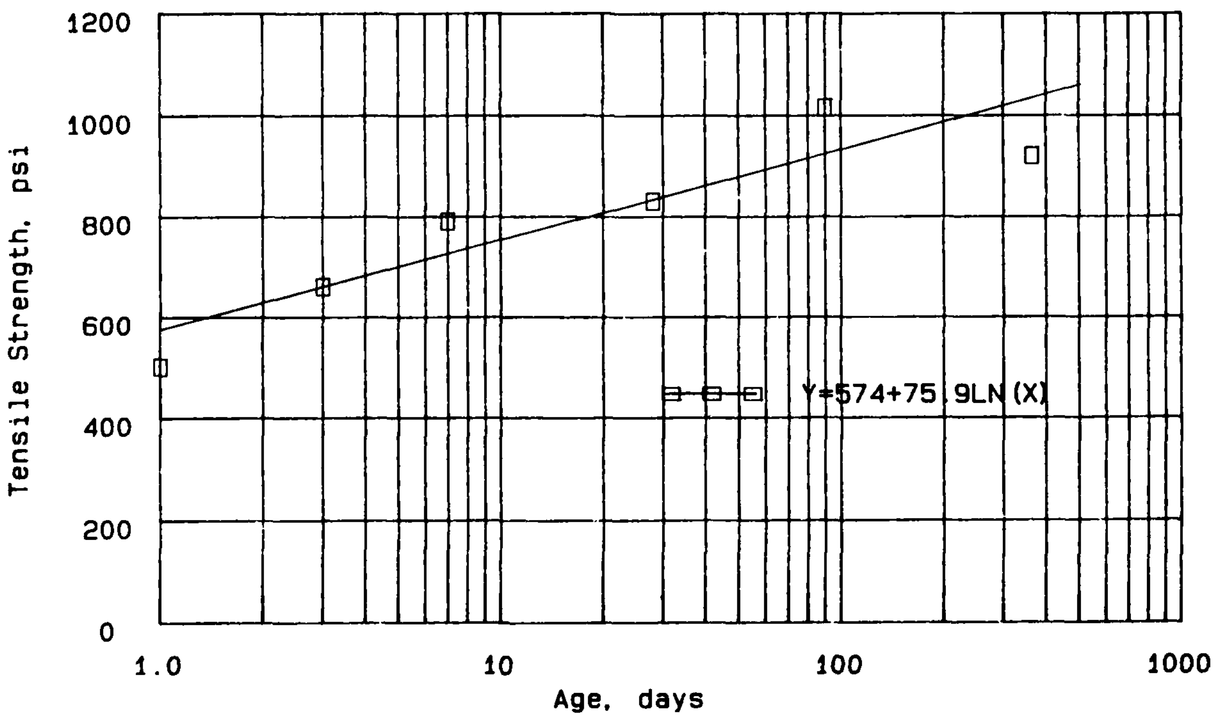

Figure 3. Splitting tensile strength gain

Poisson's ratio at 50 percent of ultimate compressive strength (Table 1).

25. The modulus of elasticity increased with age (Figure 4), ranging from 4.1×10^6 psi at 1 day to 7.0×10^6 psi at 1 year. These measured values for modulus of elasticity are essentially the same as the values calculated according to the American Concrete Institute (ACI) Committee 318 (1988) equation,

$$E = 57{,}000\sqrt{f'_c}$$

where

E = modulus of elasticity, psi $\times 10^6$

f'_c = compressive strength, psi

Overall, the average difference in actual and calculated values of modulus of elasticity was less than 6 percent (Figure 5). Ozyildirim (1986) also reported that the modulus of elasticity is similar for concrete with and without silica fume. Poisson's ratio at the various test ages was essentially constant, ranging from 0.21 to 0.24.

Creep and Shrinkage

Uniaxial creep

26. Uniaxial creep tests, conducted in accordance with ASTM C 512-87 (ASTM 1987g) were initiated at 1, 3, and 7 days. In each case, two specimens were loaded incrementally to 25 percent of the ultimate compressive strength at the time of testing. Strain measurements were made immediately prior to loading, at each load increment, upon completion of loading, and periodically during the sustained loading period of approximately 1 year. Temperature and relative humidity during the tests were approximately 73° F and 50 percent, respectively. Unloaded specimens subjected to the same environmental conditions as the loaded specimens (Figure 6) were used as controls to determine volume changes under the static temperature and moisture conditions.

27. At 1 day, creep specimens No. 4 and 8 were loaded in 200-psi increments to a total sustained load of 1,520 psi. After approximately 1 year

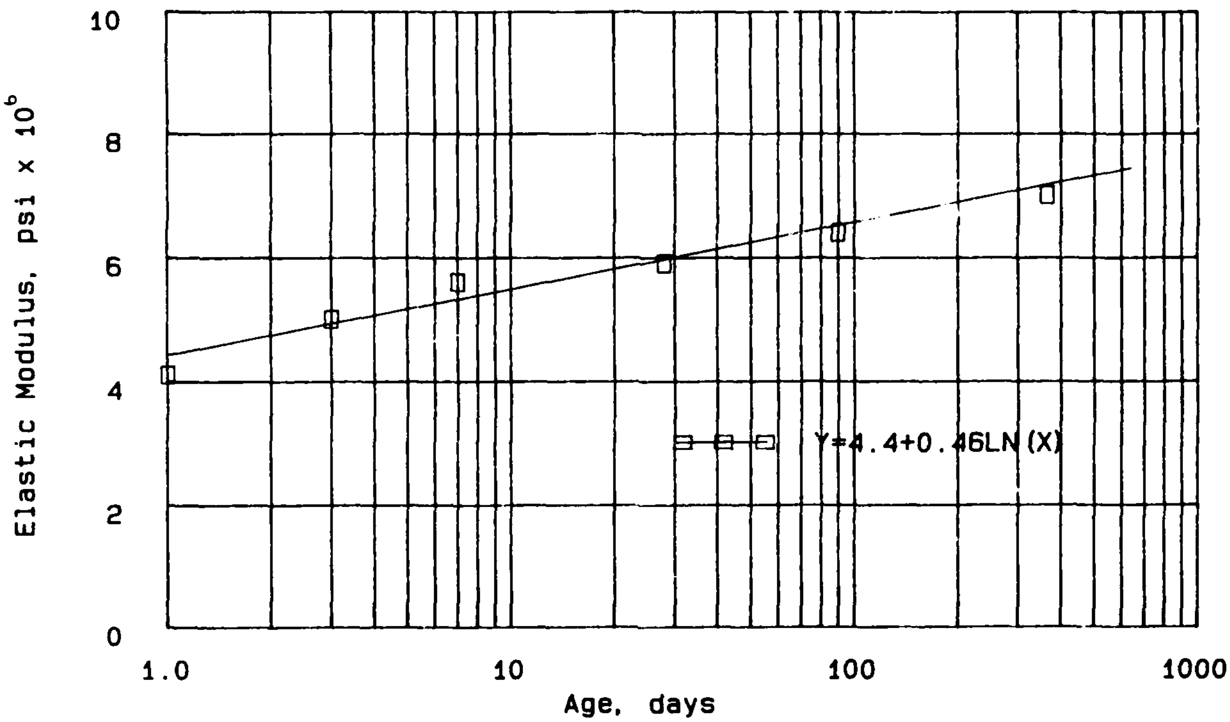

Figure 4. Increase in modulus of elasticity with time

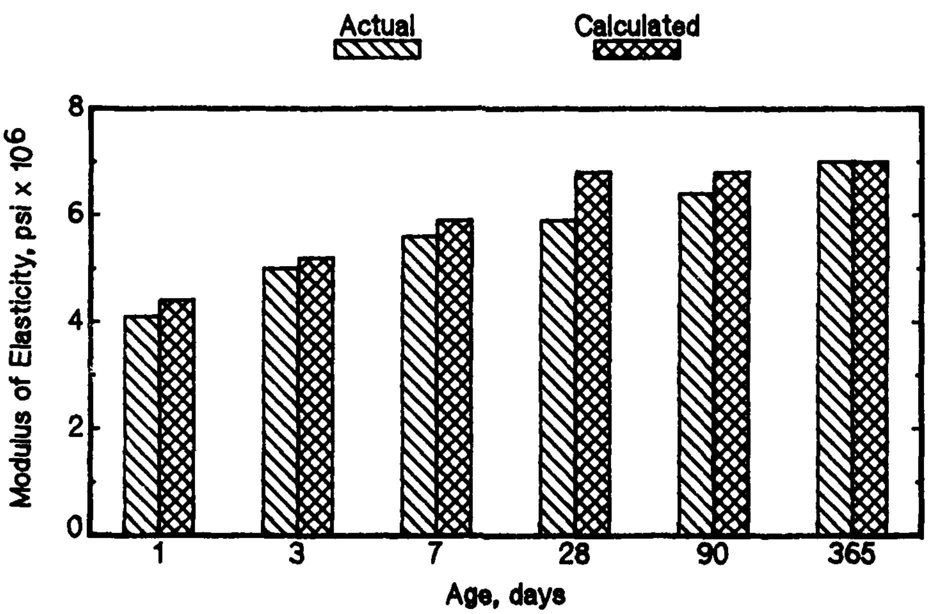

Figure 5. A comparison of actual and calculated values of modulus of elasticity

under load, the creep specimens were unloaded. Results of total strain measurements during the entire test period are presented in Figure 7.

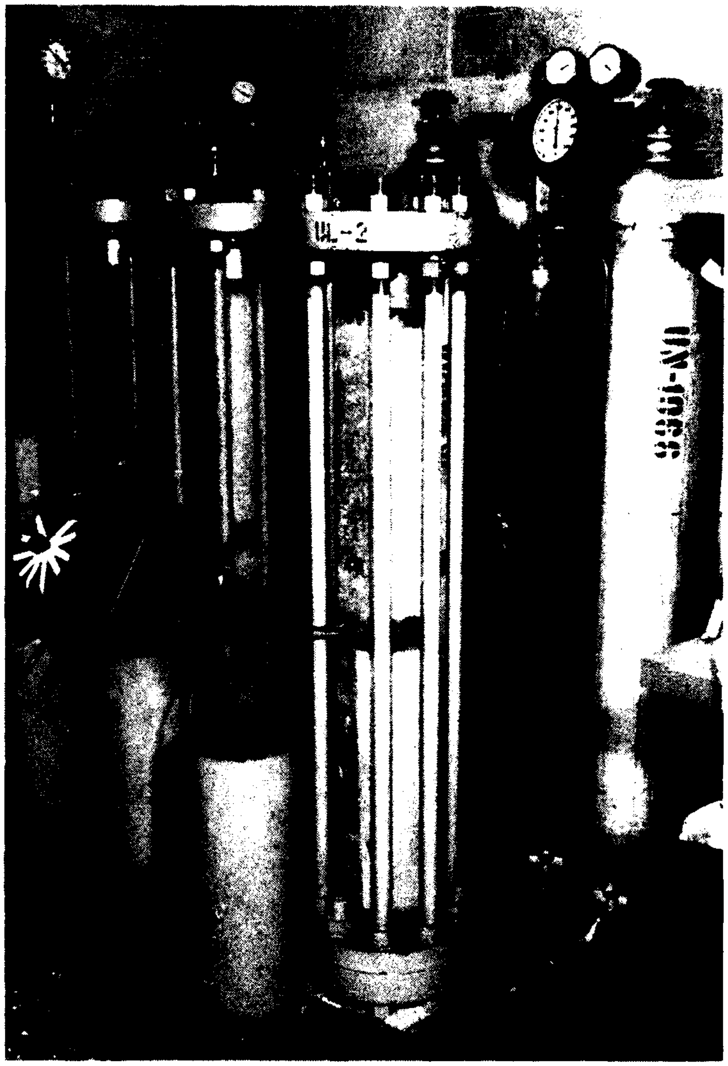

Figure 6. Creep and control specimens during testing period

Concurrently, strain measurements were made on the unloaded control specimens with results as presented in Figure 8.

28. Elastic strains resulting from application of the sustained load equal to 25 percent of the compressive strength were 320 and 331 millionths for specimens No. 4 and 8, respectively. The average secant modulus of elasticity calculated from these strains was 4.7×10^6 psi. In comparison, the average secant modulus of elasticity determined from the compressive strength tests at 50 percent of the ultimate strength was 4.1×10^6 psi. Usually, the modulus of elasticity based on the creep specimens is lower compared to that based on compressive specimens. This lower modulus is

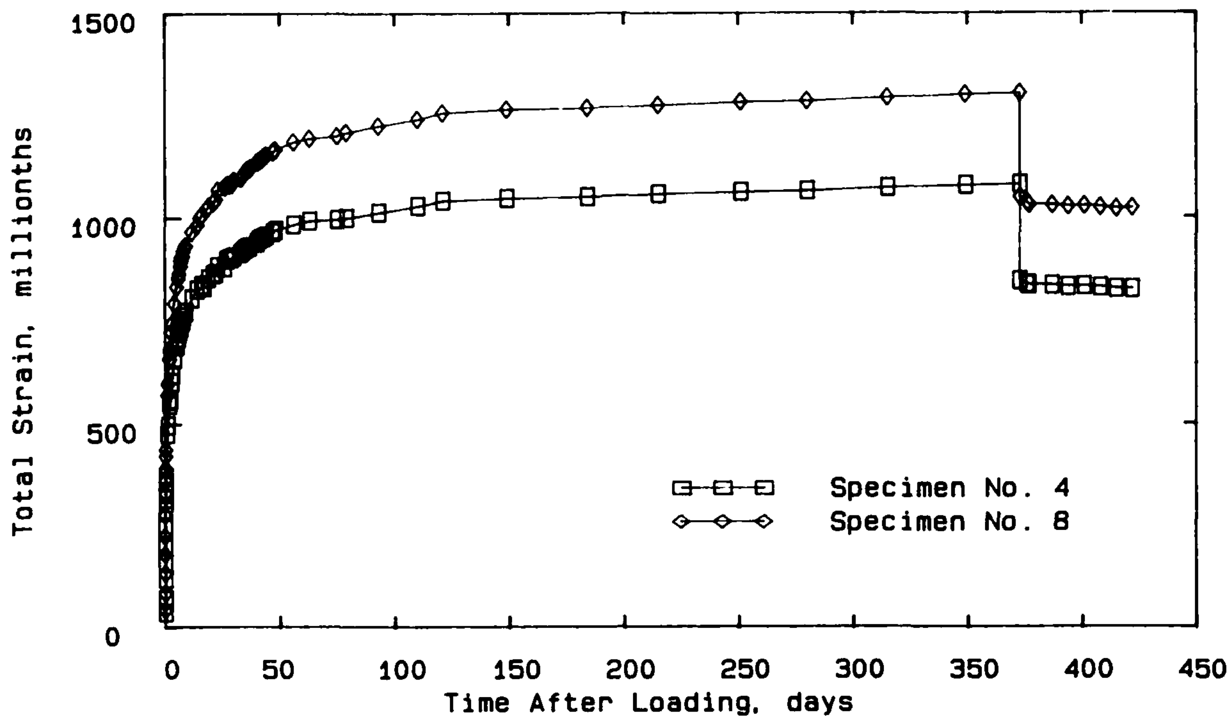

Figure 7. Total strain measured on creep specimens loaded at 1 day

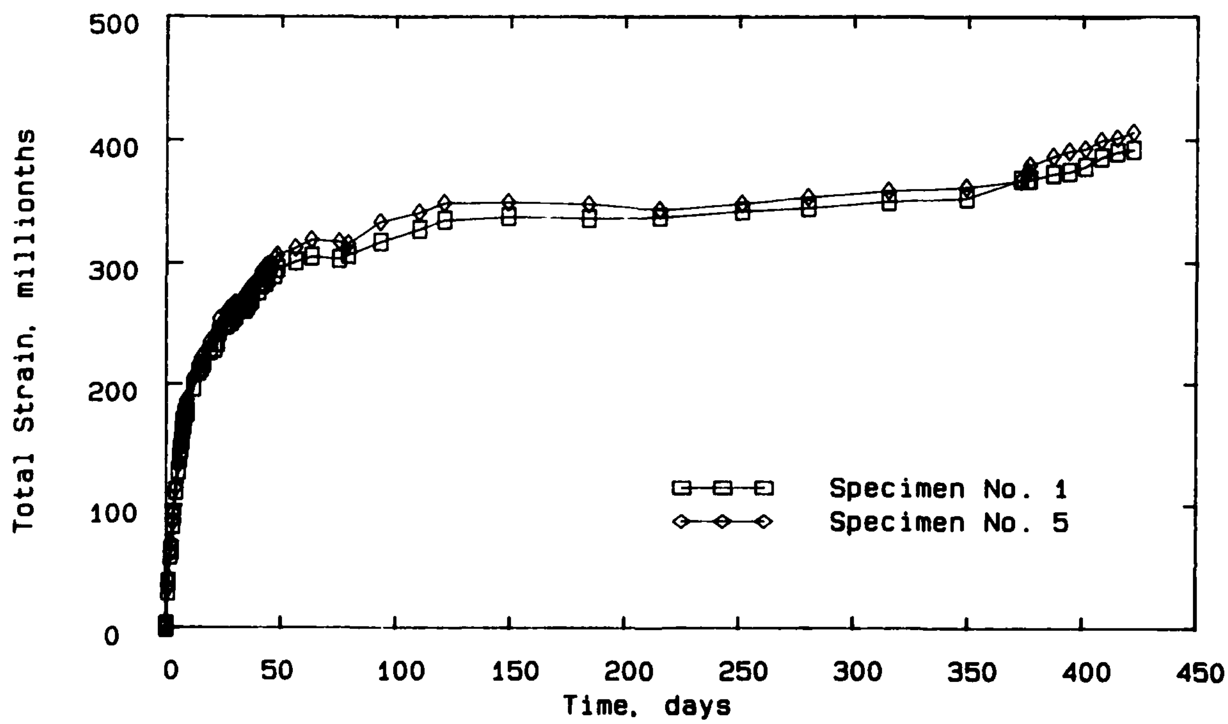

Figure 8. Results of strain measurements on unloaded control specimens

generally attributed to larger strains in the creep specimens resulting from the longer time required for load application and strain measurement. However, in this case, the modulus of elasticity based on the creep specimens was higher than that based on the compressive strength tests.

29. The elastic strain for specimen No. 8 was only 4 percent higher than that of specimen No. 4. However, at the end of the loading period, the total strain for specimen No. 8 was 20 percent higher than for specimen No. 4. Creep strains were obtained by (a) subtracting the elastic strain from the total strains for each specimen and (b) correcting this result for the appropriate volume change. Specific creep was calculated by dividing creep strains by the sustained load. Curves-of-best-fit based on least-squares analyses were computed for specific creep with results as shown in Figure 9.

30. At 3 days, specimens No. 3 and 9 were loaded in 200-psi increments to a total load of 2,050 psi. Results of total strain measurements are presented in Figure 10. Results of strain measurements on an unloaded control specimen are presented in Figure 11.

31. The elastic strain and the total strain at the end of the sustained loading period were 6 percent higher for specimen No. 3 than for specimen No. 9. The average modulus of elasticity calculated from the creep specimen data (5.1×10^6 psi) was essentially the same as that calculated from the compressive strength tests (5.0×10^6 psi). A curve-of-best-fit based on a least-squares analysis was computed for the average specific creep of the two test specimens with results shown in Figure 12.

32. At 7 days, specimens No. 6 and 10 were loaded in 200-lb increments to a total load of 2,695 psi. Results of total strain measurements are presented in Figure 13. Results of strain measurements on an unloaded control specimen are presented in Figure 14.

33. The elastic strain and the total strain at the end of the loading period were approximately 7 and 3 percent higher, respectively, for specimen No. 10 than for specimen No. 6. The average modulus of elasticity calculated from the creep specimen data (5.7×10^6 psi) was essentially the same as that calculated from the compressive strength tests (5.6×10^6 psi). A curve-of-best-fit based on a least-squares analysis was computed for the average specific creep of the two test specimens with results as shown in Figure 15.

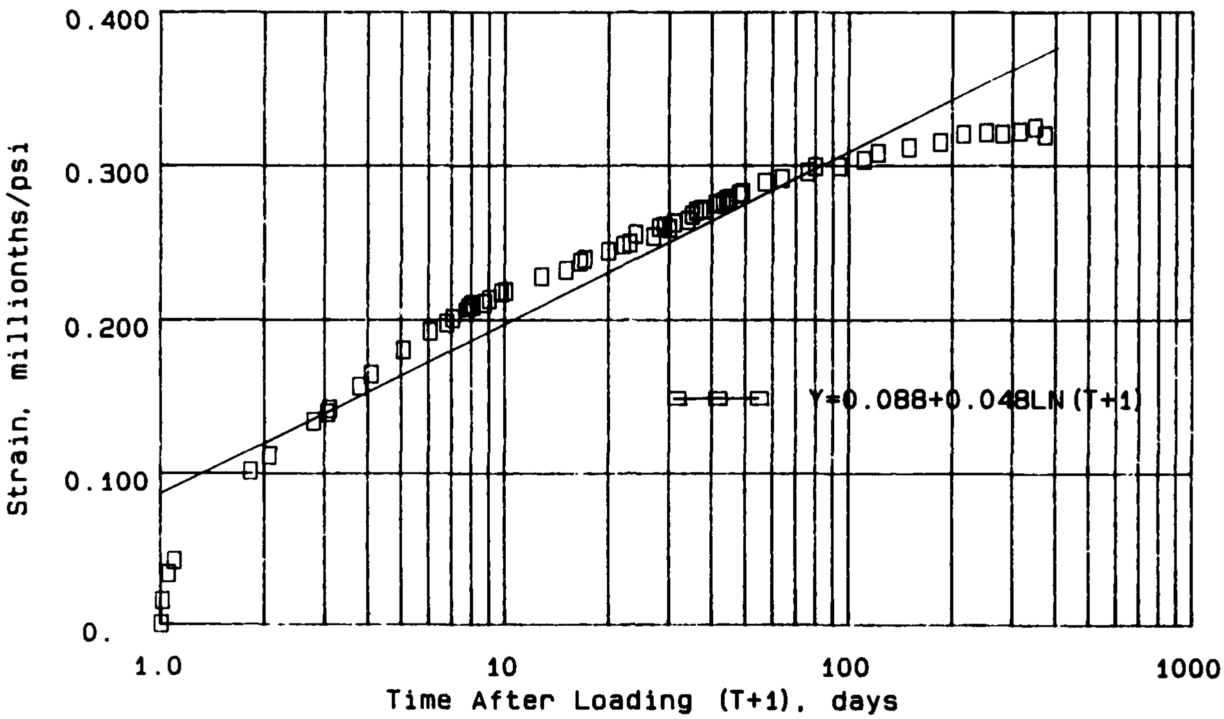

Figure 9. Average specific creep for specimens loaded at 1 day

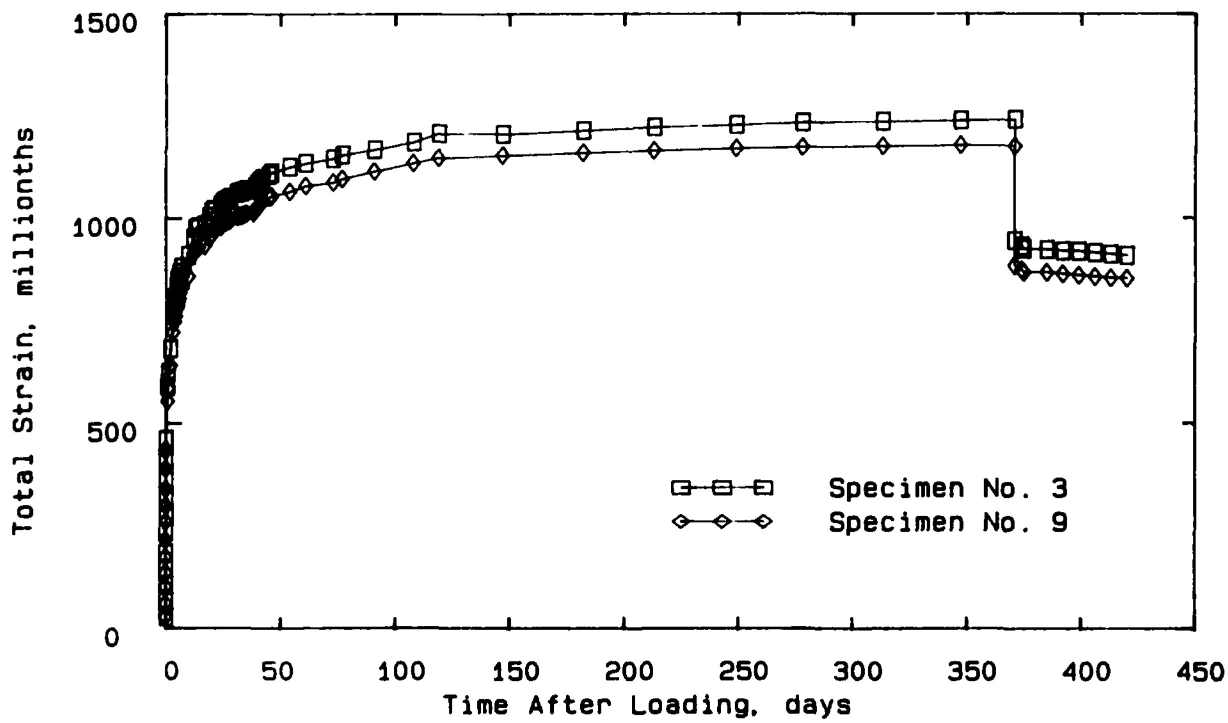

Figure 10. Total strain measured on creep specimens loaded at 3 days

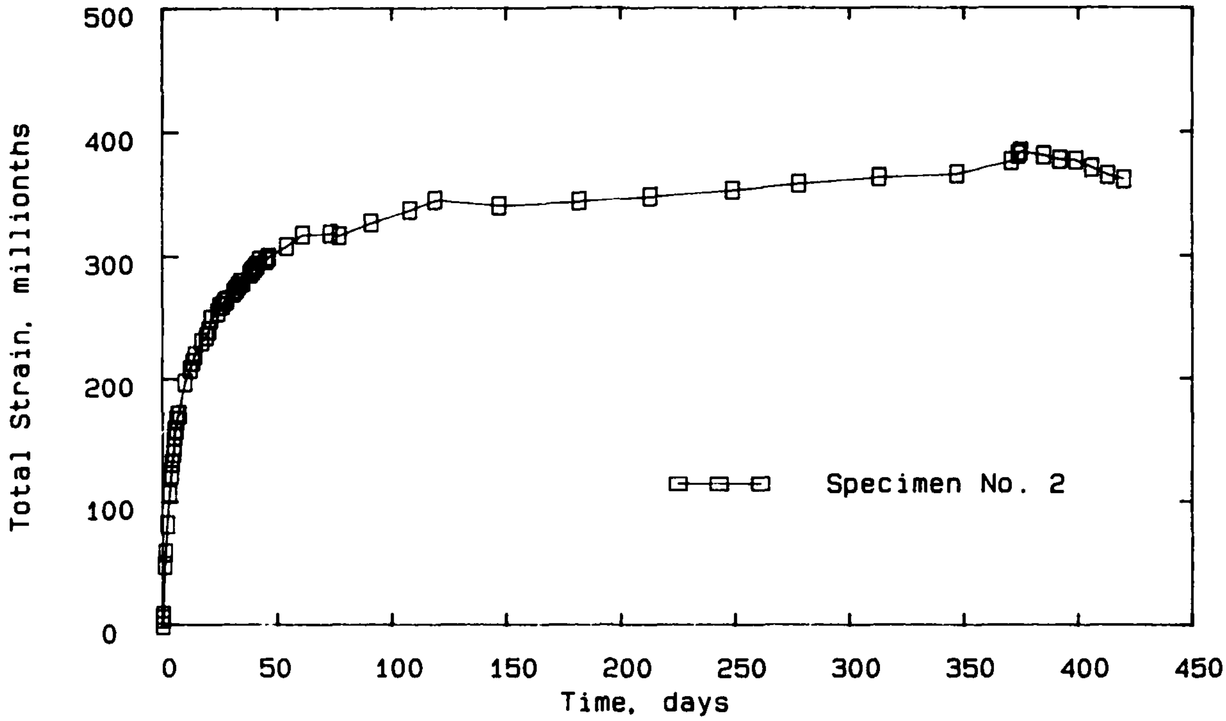

Figure 11. Results of strain masurements on an unloaded control specimen

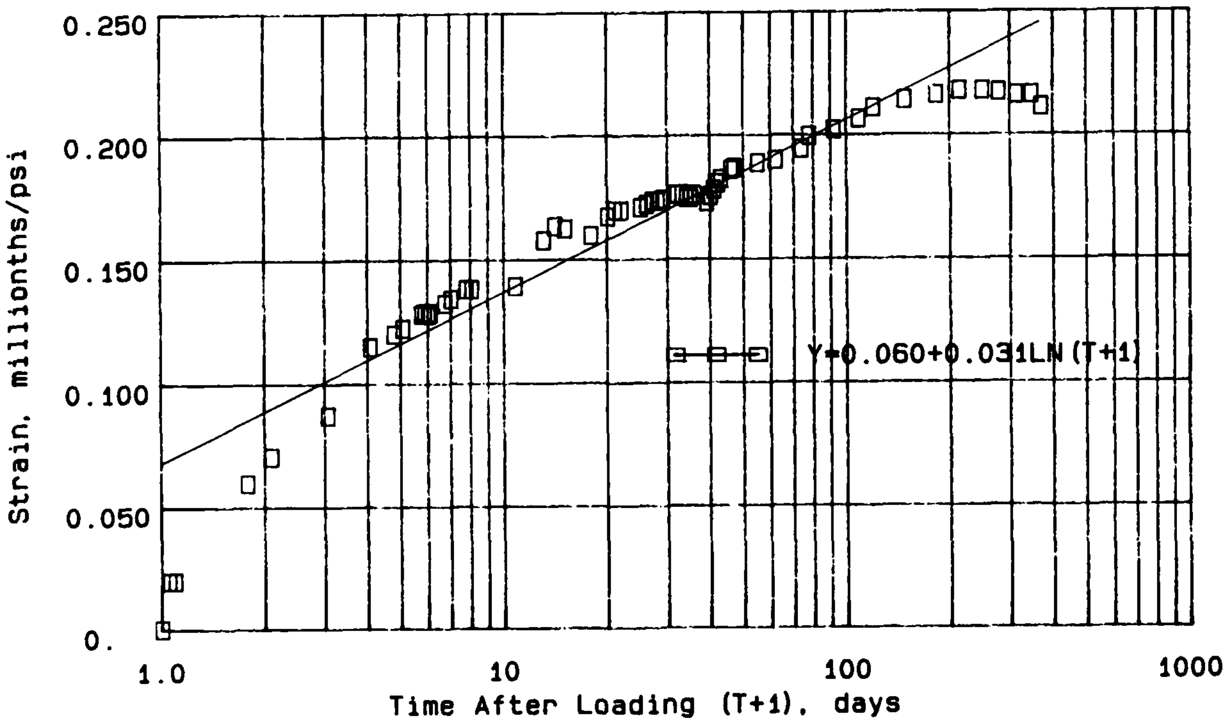

Figure 12. Average specific creep for specimens loaded at 3 days

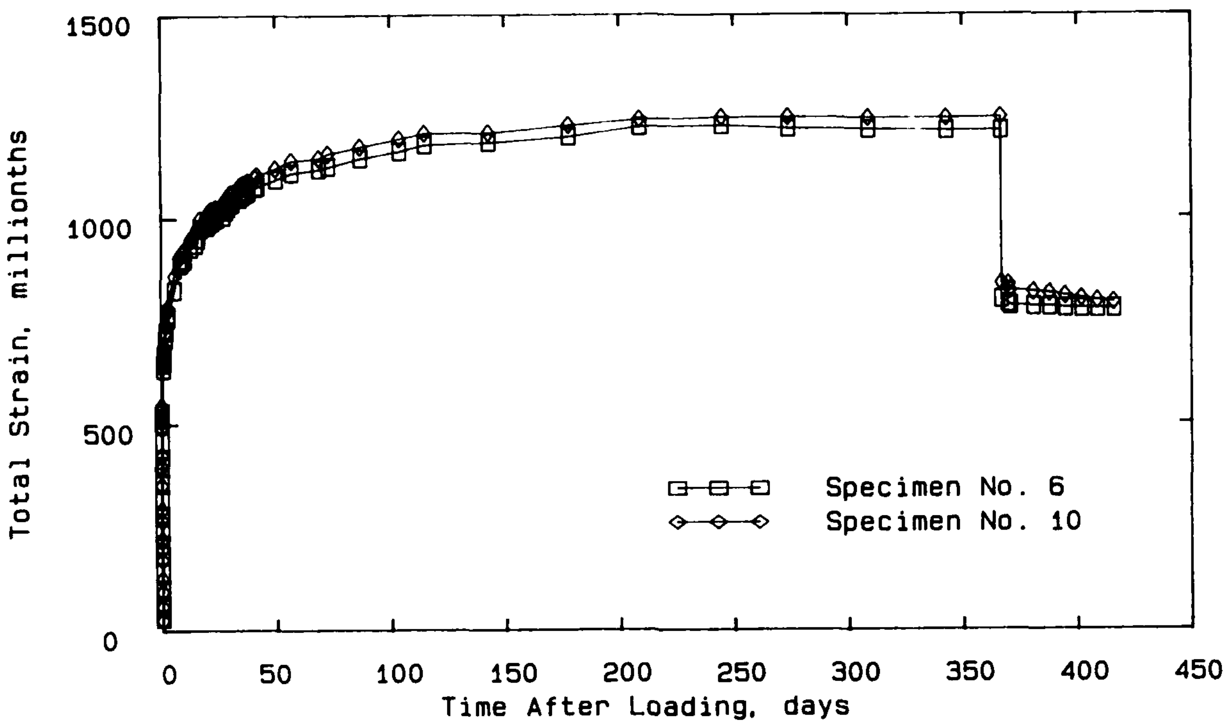

Figure 13. Total strains measured on creep specimens loaded at 7 days

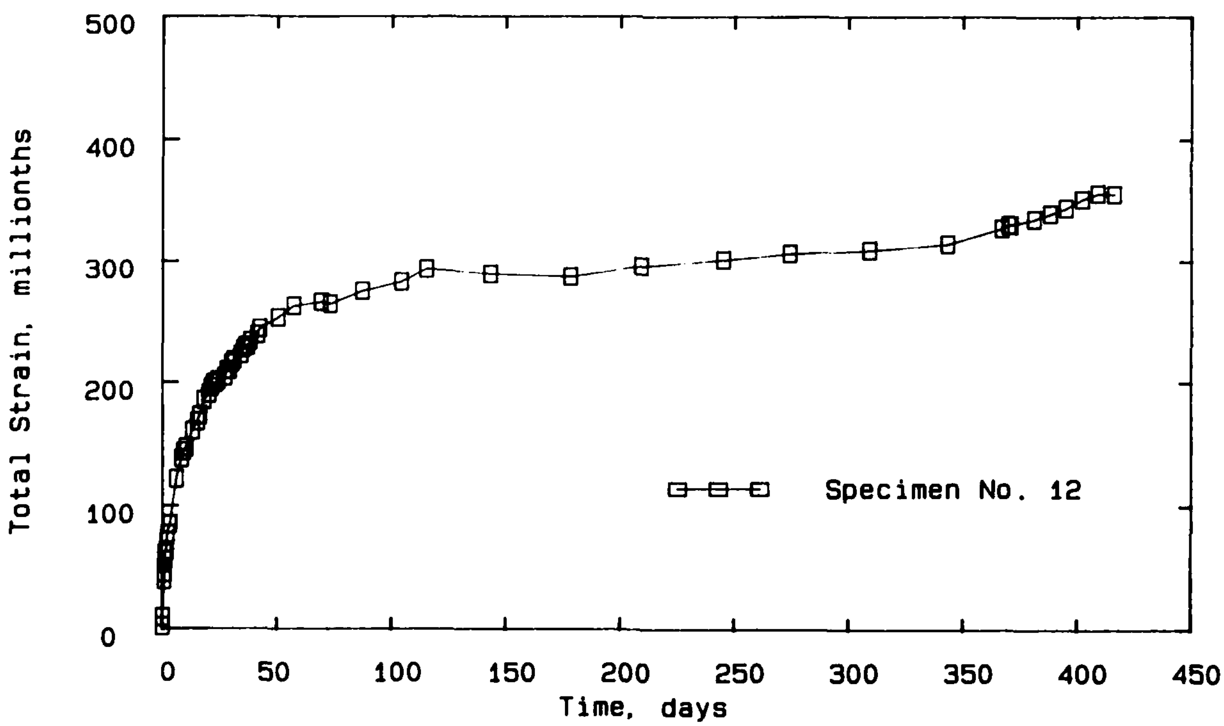

Figure 14. Results of strain measurements on an unloaded control specimen

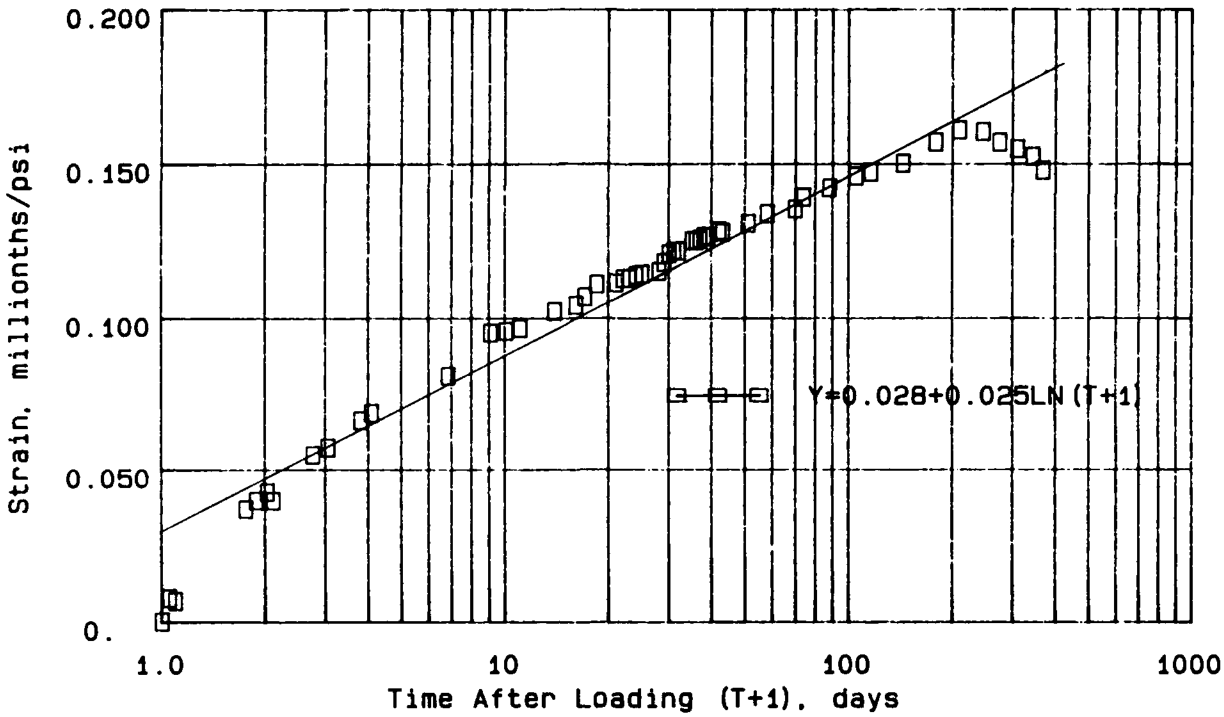

Figure 15. Average specific creep for specimens loaded at 7 days

34. The creep specimens loaded at 7 days exhibited a specific creep of 0.14 millionths/psi after 90 days under a sustained load of 2,695 psi. Saucier (1984) reported a specific creep of 0.086 millionths/psi for silica-fume concrete specimens loaded to 2,000 psi at 28 days. Also, specific creep of the same concrete mixture without silica fume was 0.098 millionths/psi or about 12 percent higher, under the same test conditions. Considering that the earlier the concrete is loaded the greater the resulting creep, the creep test results reported herein appear to be comparable with those previously reported by Saucier (1984). Others have also reported that creep of silica-fume concrete is less than that of comparable conventional concrete. For example, Houde, Prezeau, and Roux (1987) reported that "creep of concrete with and without fibers was decreased by at least 20 percent when 5 to 10 percent of cement was replaced by silica fume."

35. After 1 year under a sustained load of 800 psi, conventional concretes exhibited creep strains ranging from about 450 to 1,150 millionths depending on the type of aggregate (Troxell, Raphael, and Davis 1958). These 4- by 14-in. cylindrical specimens were loaded at 28 days and stored in air at 70° F and 50-percent relative humidity after loading. The specific creep of

these concretes ranged from 0.56 to 1.44 millionths/psi or 3 to 8 times higher than the creep of silica-fume concrete loaded at 7 days in the tests reported herein.

Drying shrinkage

36. Strain measurements on the unloaded control specimens in the creep tests were used to determine the drying shrinkage of silica-fume concrete exposed to 73° F and 50 percent relative humidity. Tests were initiated at 1, 3, and 7 days with results as shown in Figure 16.

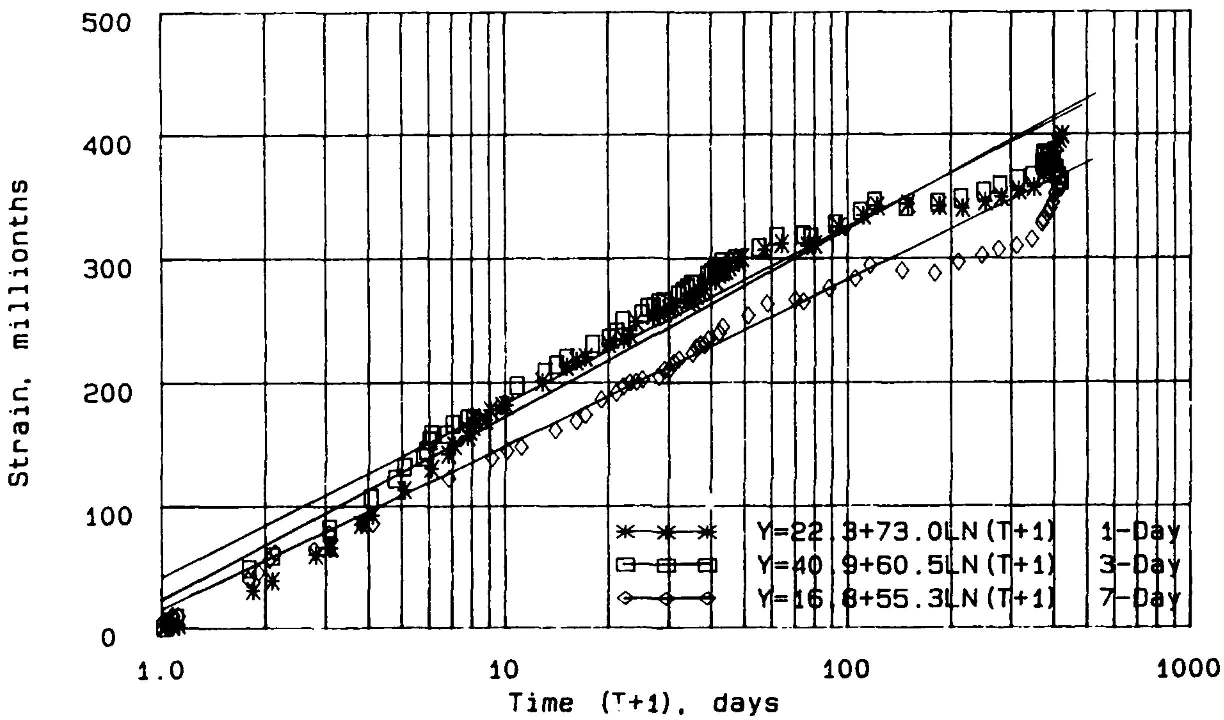

Figure 16. Results of drying shrinkage tests initiated at 1, 3, and 7 days

37. After 1 year of exposure in air, the drying shrinkage of silica-fume concrete was 400, 398, and 343 millionths for tests initiated at 1, 3, and 7 days, respectively. In similar tests initiated at 28 days, 4- by 14-in. cylinders of conventional concrete exhibited drying shrinkage strains ranging from 400 to 1,000 millionths depending on the type of aggregate (Troxell, Raphael, and Davis 1958).

38. In tests initiated at 14 days, Saucier (1984) reported drying shrinkage strains of 335 and 380 millionths for concrete with and without silica fume, respectively, after 90 days storage in air. In comparison, the current tests initiated at 1, 3, and 7 days resulted in drying shrinkage

strains of 311, 314, and 266 millionths, respectively, after 90 days storage in air. These test results appear to be in general agreement with others who have reported that silica-fume concrete has "similar but lower drying shrinkage values" as compared with conventional concrete (Ozyildirim 1986). Tazawa and Yonekura (1986) also reported that the drying shrinkage of silica-fume concrete was lower than that of concrete without silica fume at the same w/c ratio. However, they noted that in the case of standard curing, the values of drying shrinkage per unit cement paste volume were roughly the same for both concretes with and without silica fume at the same compressive strength.

Autogenous shrinkage

39. Periodic strain measurements on four test specimens continuously stored in a moist curing room were used to determine the autogenous shrinkage of silica-fume concrete. Results of these tests are presented in Figure 17. The time of setting of the concrete, determined according to ASTM C 403-85 (ASTM 1987c), was 13.25 hr after casting. Strain and temperature measurements corresponding to the time of setting were used as the zero point for calculating the autogenous shrinkage.

40. Average results of the autogenous shrinkage tests and the drying shrinkage tests initiated at 1 day are compared in Figure 18. Strain measurements under the two test conditions followed a similar trend, increasing shrinkage at a decreasing rate, for approximately the first 150 days. Drying shrinkage strains were about three times greater than the autogenous values at this point. Beyond this time, all autogenous volume-change specimens exhibited expansion which reduced the net shrinkage to an average of 31 millionths at the end of the test period. Limited measurements by Saucier (1984) on 3-in. expansion bars stored in water appear to exhibit this same trend on an accelerated time scale (Figure 19). In these tests, concrete with and without silica fume exhibited autogenous shrinkage to 28 days and expansion beyond this age.

41. Average results of the autogenous shrinkage tests at early ages (Figure 20) exhibit a rapid shrinkage of about 60 millionths within the first 24 hr followed by a more gradual increase in shrinkage to approximately 80 millionths at 10 days. Similar results were reported by Paillere, Buil, and Serrano (1989) who noted that whereas conventional concrete shrinks very slowly after the usual stage of hydration swelling, silica-fume concrete

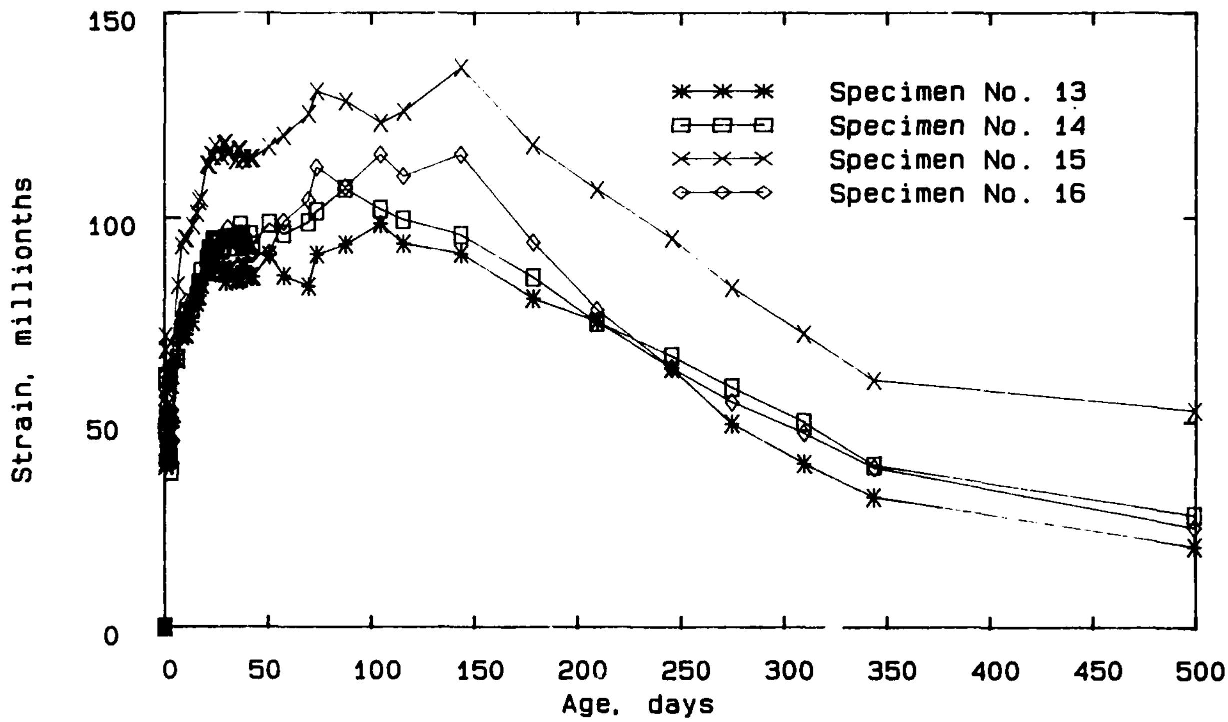

Figure 17. Autogenous shrinkage of silica-fume concrete

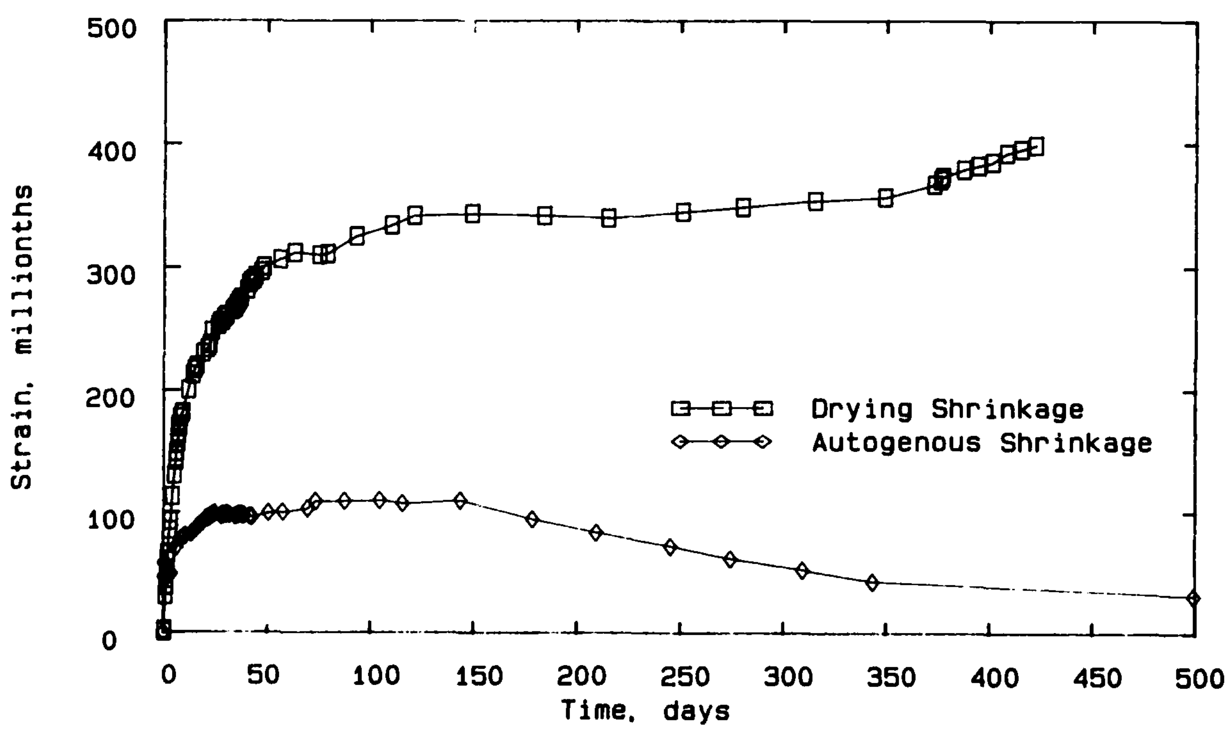

Figure 18. Average results of drying shrinkage tests initiated at 1 day and autogenous shrinkage tests

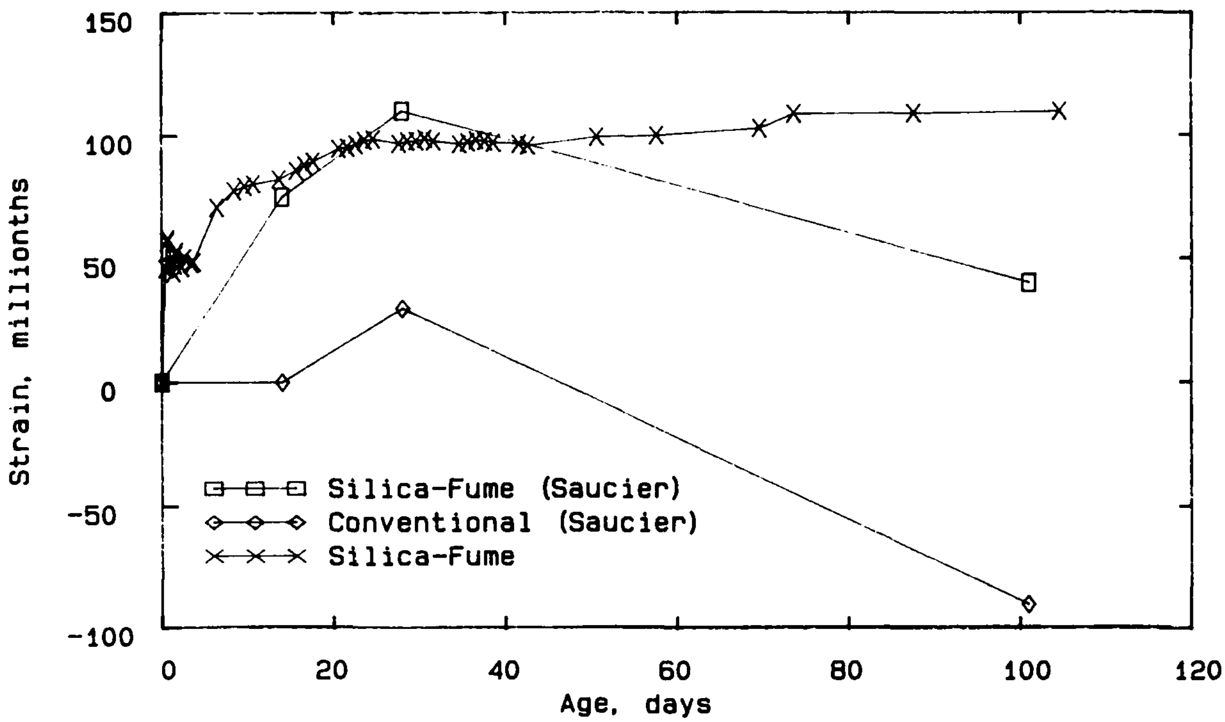

Figure 19. Autogenous volume change of silica-fume concrete compared to conventional concrete

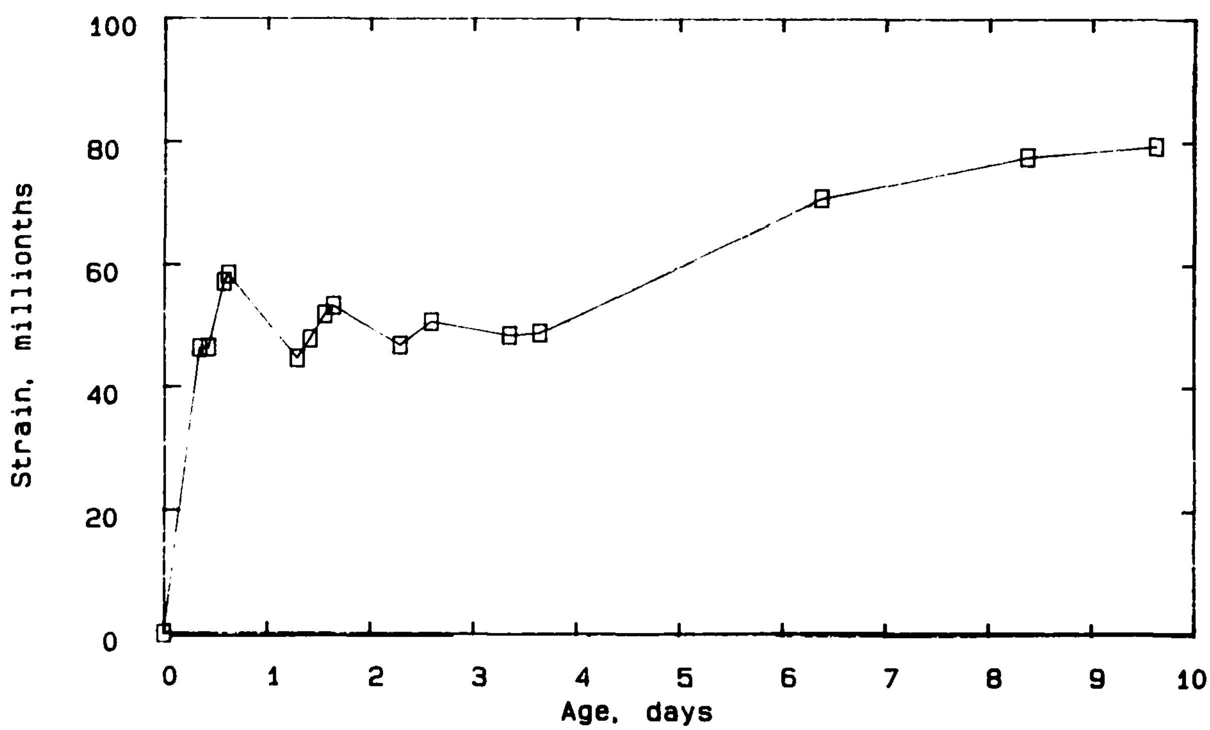

Figure 20. Average results of autogenous shrinkage tests at early ages

(w/c = 0.26) exhibited no swelling stage and shrinkage was immediate (from 6 hr) reaching 105 millionths at 92 hr. Also, in the absence of any evaporation or desiccation, concrete having very low w/c ratios, with and without silica fume, exhibits high autogenous shrinkage sufficient to cause cracking. This high autogenous shrinkage is attributed to self-desiccation resulting from the hydration of the cement at a very low w/c ratio. Attempts to correct this weakness of the material by adding steel fibers were only partially successful (Paillere, Buil, and Serrano 1989). The fiber-reinforced concretes exhibited an autogenous shrinkage lower than the reference concrete but still cracked at a later age under restrained deformation.

42. Periodic length-change measurements on expansion bar specimens (3-in. square prisms with a gage length of 10 in.) were used to determine the restrained autogenous volume change of silica-fume concrete. During the testing period, these specimens were continuously stored in three different environments, each one at 73° F: (a) plastic bags in air, (b) moist curing room, and (c) lime-saturated water. In each case, shrinkage strains were slower to develop compared to results of the unrestrained volume change tests at comparable ages (Figure 21). In fact, specimens stored in the moist curing

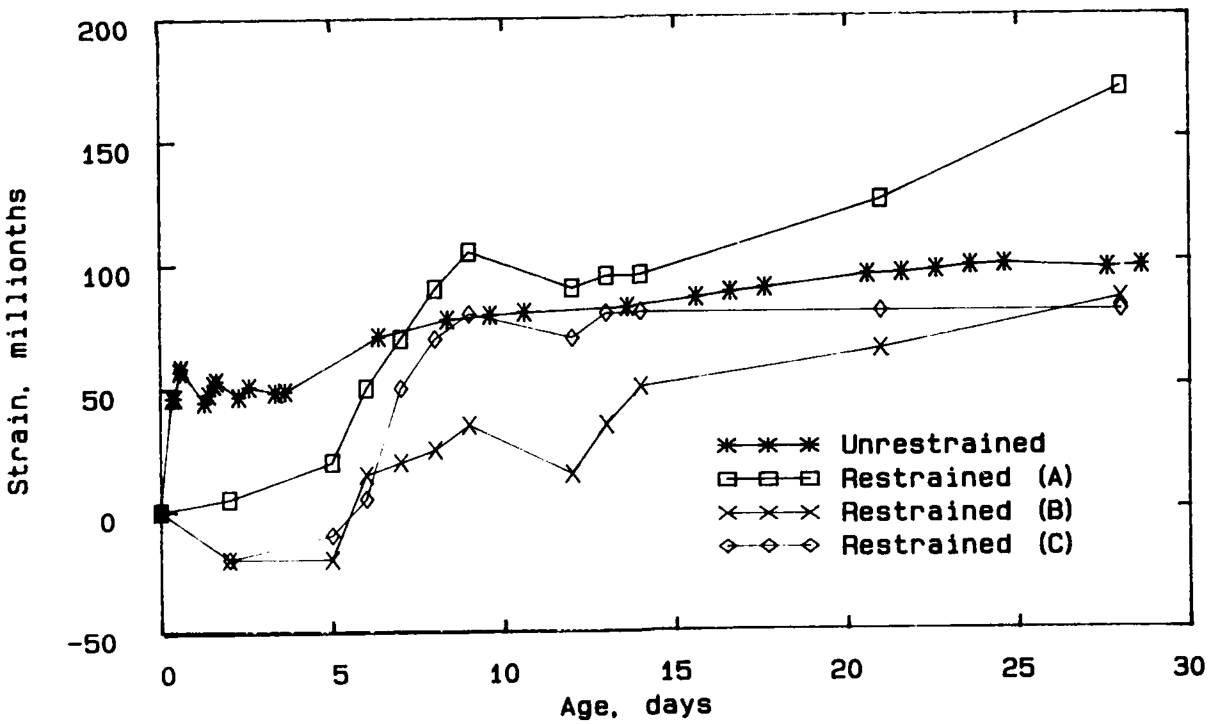

Figure 21. Autogenous volume change of unrestrained and restrained test specimens

room and lime-saturated water exhibited an average expansion strain of 15 millionths at 5 days. At this point, these specimens began to shrink and at the end of the test exhibited an average shrinkage strain of 83 millionths, 15 percent less than the unrestrained test results. The restrained specimens placed in plastic bags and stored in air (environment a) did not exhibit a swelling stage. These specimens, with no external source of moisture, exhibited immediate and continuing shrinkage with an average strain of 170 millionths, approximately twice that of the other restrained test specimens, at 28 days.

Ultimate Strain Capacity

43. Ultimate strain capacity tests, performed in accordance with CRD-C 71-80 (US Army Engineer Waterways Experiment Station (USAEWES) (1949c)) were conducted at 1, 3, 7, 28, 90, 180, and 365 days. At each age, two beams were loaded to failure in third-point flexure with a loading rate of 40 psi/min outer fiber stress. In the early-age tests, strains were recorded manually after each 1,000-lb increment of total load. In later tests, load and strain data were continuously recorded on magnetic tape.

44. The actual modulus of rupture was calculated for each specimen in accordance with ASTM C 78-84 (ASTM 1987b). Results are presented in Table 2. The actual and predicted values for modulus of rupture are compared in Figure 22. The predicted values were calculated according to the ACI Committee 318 (1988) equation,

$$f_r = 7.5\sqrt{f'_c}$$

where

f_r = modulus of rupture, psi

f'_c = compressive strength, psi

The differences in actual and calculated values were less than 10 percent at 1 and 3 days. However, the differences increased significantly at compressive strengths in excess of 10,000 psi. The actual modulus of rupture exceeded the calculated by an average of 23 percent in tests beyond 3 days.

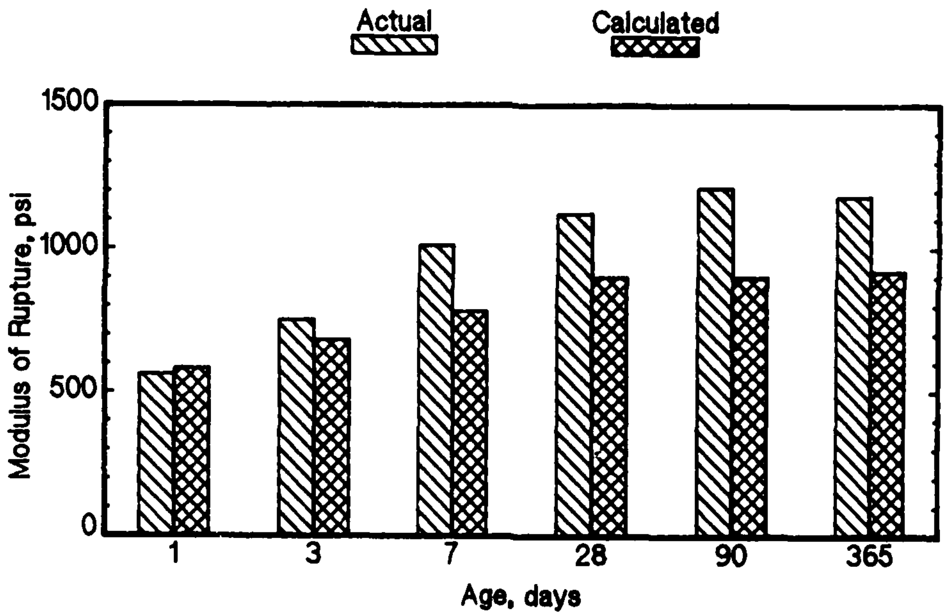

Figure 22. Comparison of actual and calculated values for modulus of rupture

45. Strains measured at the strain meters were extrapolated to the surface fibers of the specimens assuming a linear strain distribution. Typical load-strain curves are presented in Figure 23. The ultimate tensile strain capacity is the tensile strain at 90 percent of the ultimate load. Ultimate strain capacity test data are presented in Table 2.

46. The higher tensile stress and strain capacities of silica-fume concrete compared with that of conventional concrete used in recently constructed Corps of Engineers projects (Bombich, Sullivan, and McDonald 1977; Holland, Liu, and Bombich 1982) are shown in Figure 24. The tensile strain capacity of the silica-fume concrete averaged about 2-1/2 times greater than that of conventional concrete. The ratios between stress and strain capacities accurately reflect the higher modulus of elasticity of the silica-fume concrete.

Thermal Properties

Adiabatic temperature rise

47. The adiabatic-heat-rise test was initiated in accordance with CRD-C 38-73 (USAEWES 1949a). However, the test was terminated prematurely

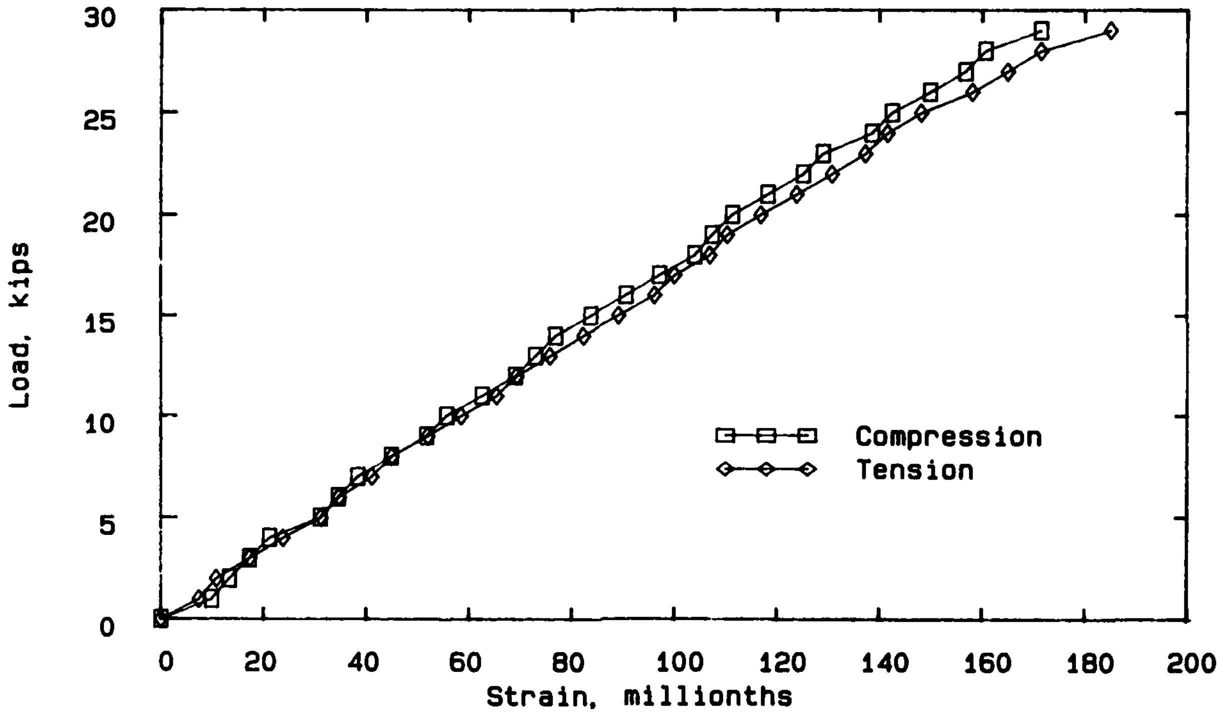

Figure 23. Typical load-strain curves determined from ultimate strain capacity tests at 7 days

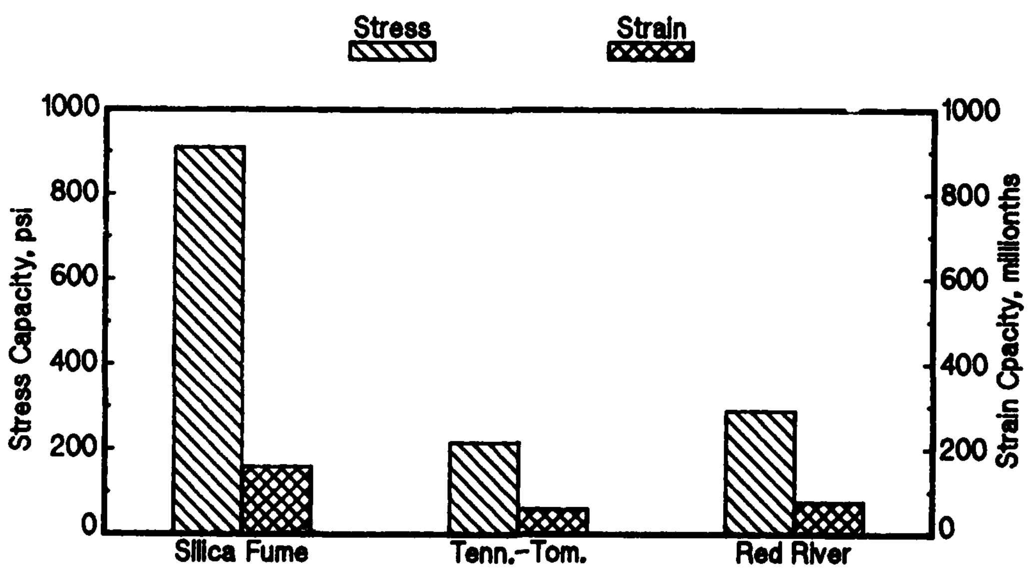

Figure 24. Tensile stress and strain capacities of silica-fume concrete compared with that of conventional concrete

because of equipment failure. The results obtained (Table 3) indicate a temperature rise for silica-fume concrete of 97.1° F after approximately 8-1/2 days. This significantly higher temperature rise (Figure 25), compared with that for conventional concrete with a w/c of 0.50 (Holland, Liu, and Bombich 1982), is primarily attributed to the increased cement content of the concrete containing silica fume.

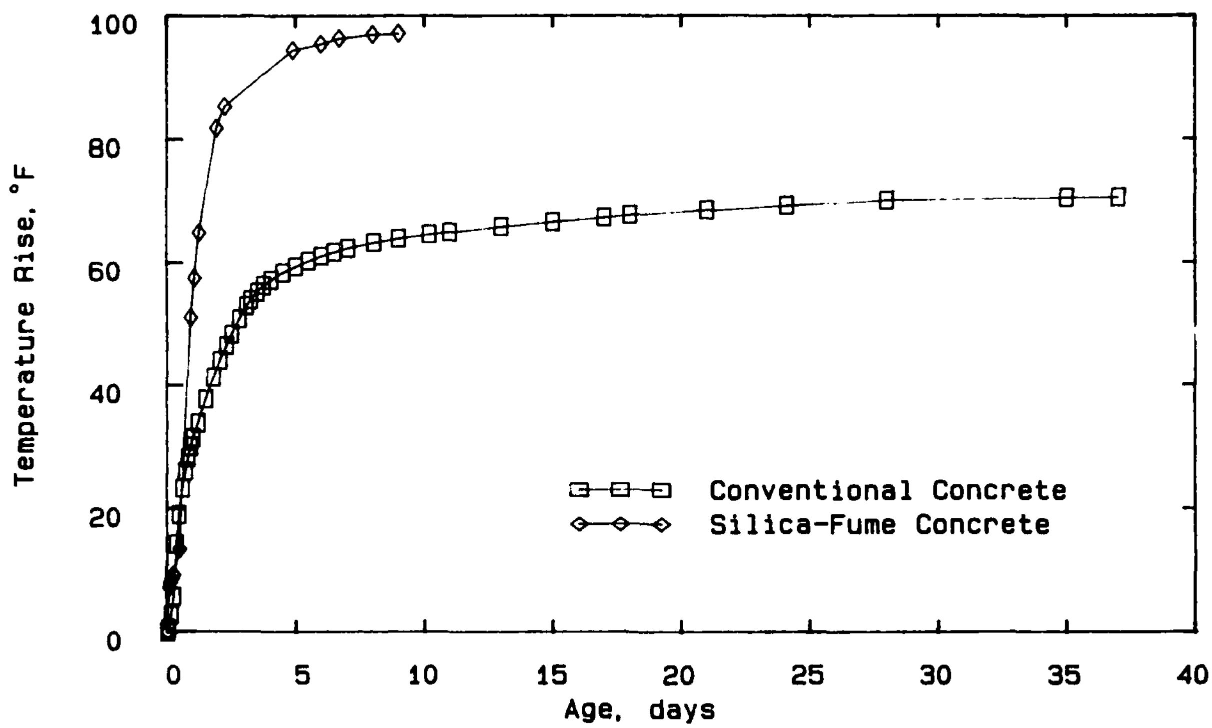

Figure 25. Adiabatic temperature rise for conventional and silica-fume concrete

48. Results of previous tests on conventional concrete mixtures for recently constructed Corps hydraulic structures indicate a linear relationship between temperature rise and cement content (Figure 26). Based on these tests, the predicted temperature rise for the silica-fume concrete (650 lb of cement) is 91° F. The measured temperature rise was 97.1° F after 8-1/2 days, and had the test been completed, it is expected that a temperature rise of at least 100° F would have been recorded. Such a temperature rise would have been approximately 10 percent higher than predicted for conventional concrete with the same cement content. Similar results were reported by Maage (1986) who found that adding 10 percent silica fume by weight of cement increased the

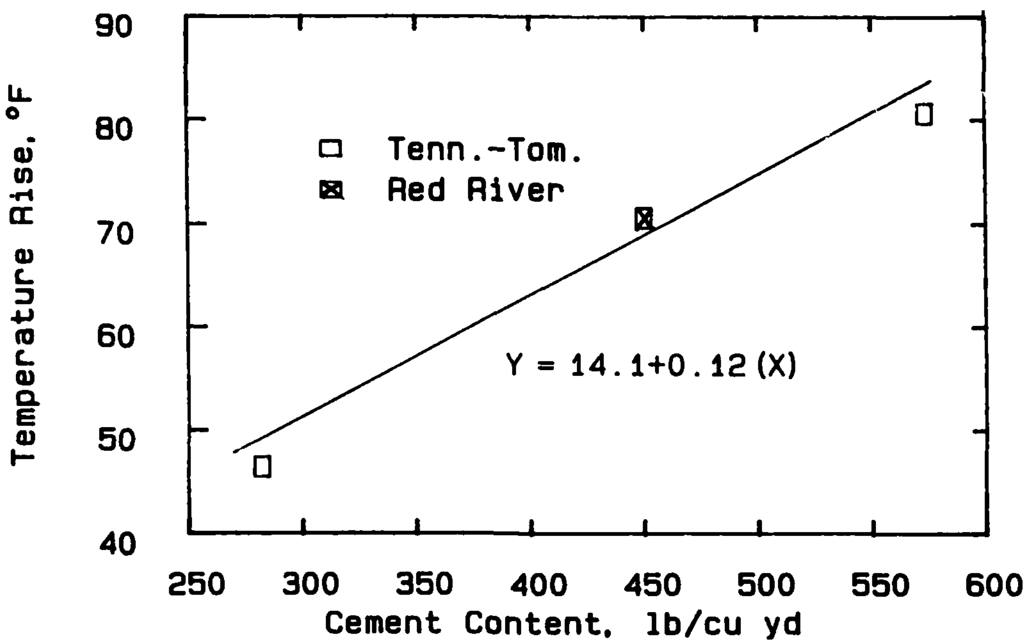

Figure 26. Relationship between cement content and adiabatic temperature rise of conventional concrete mixtures

adiabatic temperature rise by about 8 percent compared to a conventional concrete mixture proportioned for the *same* compressive strength.

Coefficient of thermal expansion

49. The coefficient of thermal expansion, determined in accordance with CRD-C 39-81 (USAEWES 1949d), was 6.7 millionths/°F. This value is similar to those values previously determined for conventional concrete used on recently constructed Corps projects (Figure 27). Ozyildirim (1986) also reported that the thermal coefficients of expansion for concrete with and without silica fume were "comparable, indicating that the addition of silica fume would not have any significant effect on the thermal compatibility of overlays."

Abrasion Erosion

50. Abrasion-erosion testing was conducted in accordance with CRD-C 63-80 (USAEWES 1949b). This test procedure involves subjecting the concrete specimens to abrasion erosion caused by the wear of steel grinding balls on the concrete surface (Figure 28). The steel grinding balls are propelled by water in the test chamber. The water is in turn propelled by a

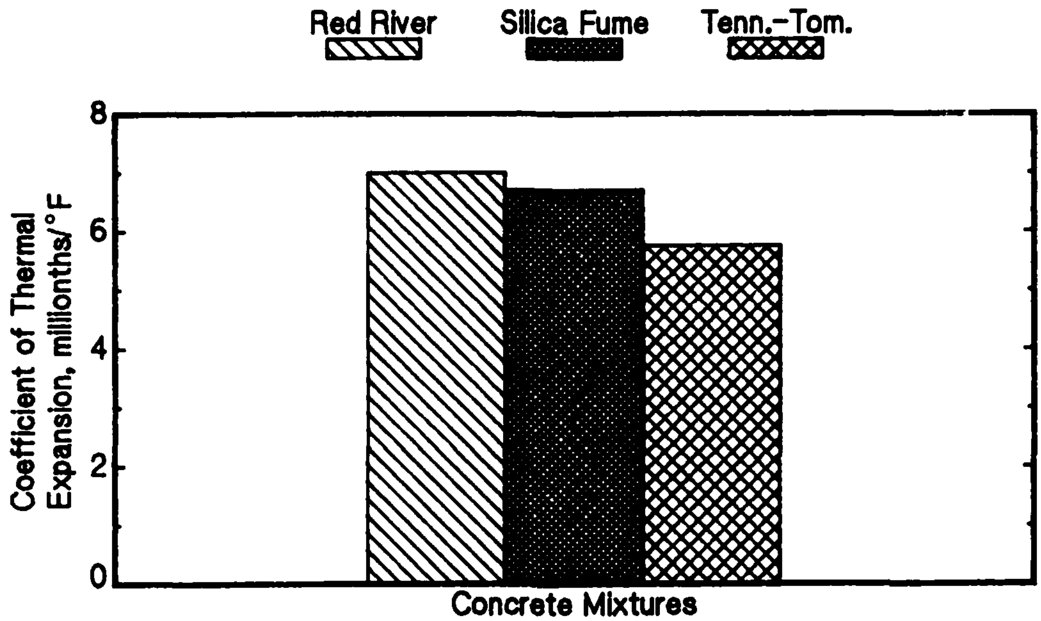

Figure 27. The coefficient of thermal expansion of silica-fume concrete compared with that of conventional concrete

submerged mixer paddle. Test specimens are periodically removed from the test apparatus to determine the amount of abrasion-erosion damage. The damage is quantified and reported as a percentage of original mass lost. The development of the test procedure and data from a large number of tests of various concrete mixtures were described by Liu (1980).

51. Abrasion-erosion testing of three specimens was initiated at 28 days with results as shown in Table 4. Abrasion-erosion losses, expressed as a percentage of original mass, ranged from 2.4 to 3.3 percent with an average loss of 2.9 percent (Figure 29). As shown in Figure 30, this average loss was approximately 60 percent less than that for concrete of similar limestone coarse aggregate which did not contain silica fume and an HRWRA (Holland 1983). This significant improvement in abrasion-erosion resistance is attributed to the increased compressive strength of the silica-fume concrete, 14,280 psi at 28 days compared to 5,710 psi for the conventional concrete. The abrasion-erosion resistance of the silica-fume concrete is similar to that of a conventional concrete mixture (0.40 w/c ratio) containing a very hard chert aggregate (Figure 30). The chert aggregate concrete had a compressive strength at 28 days of 9,020 psi (Liu 1980).

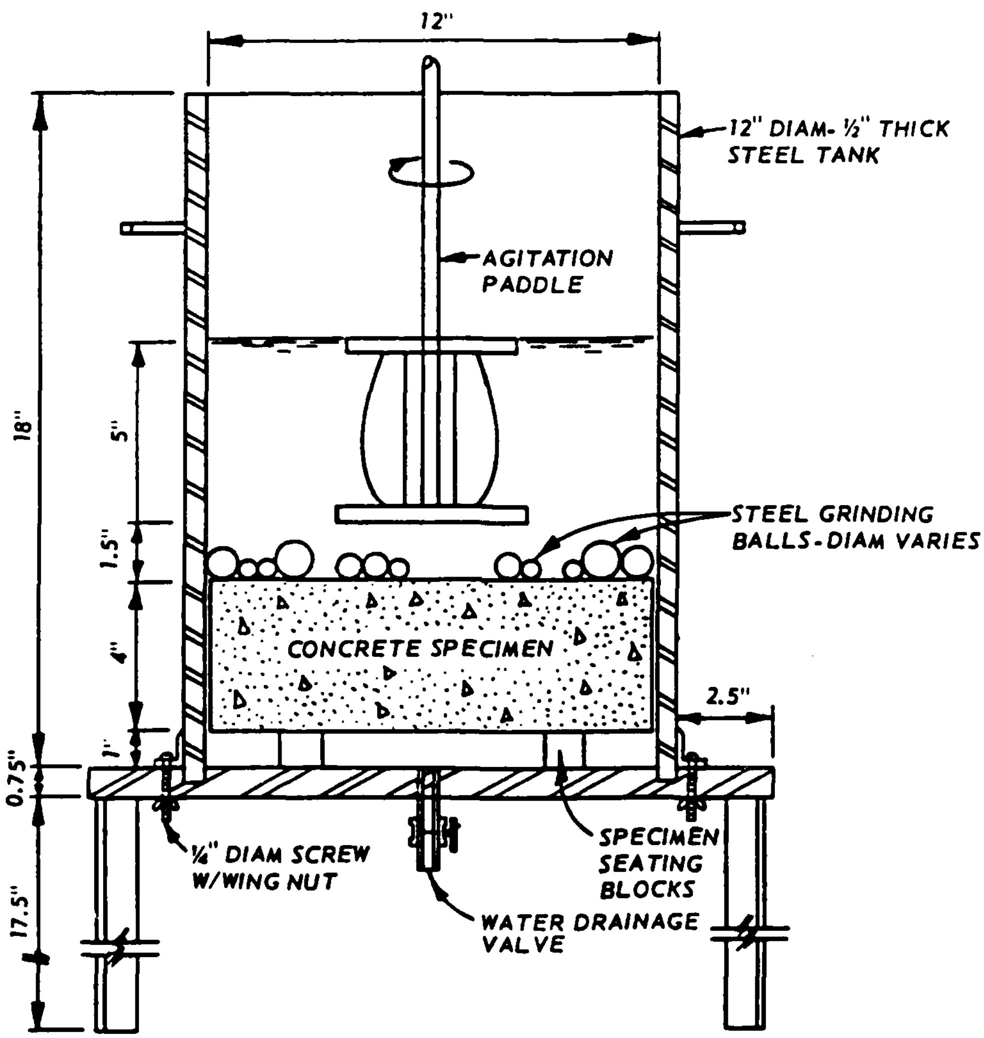

Figure 28. Abrasion-erosion test apparatus

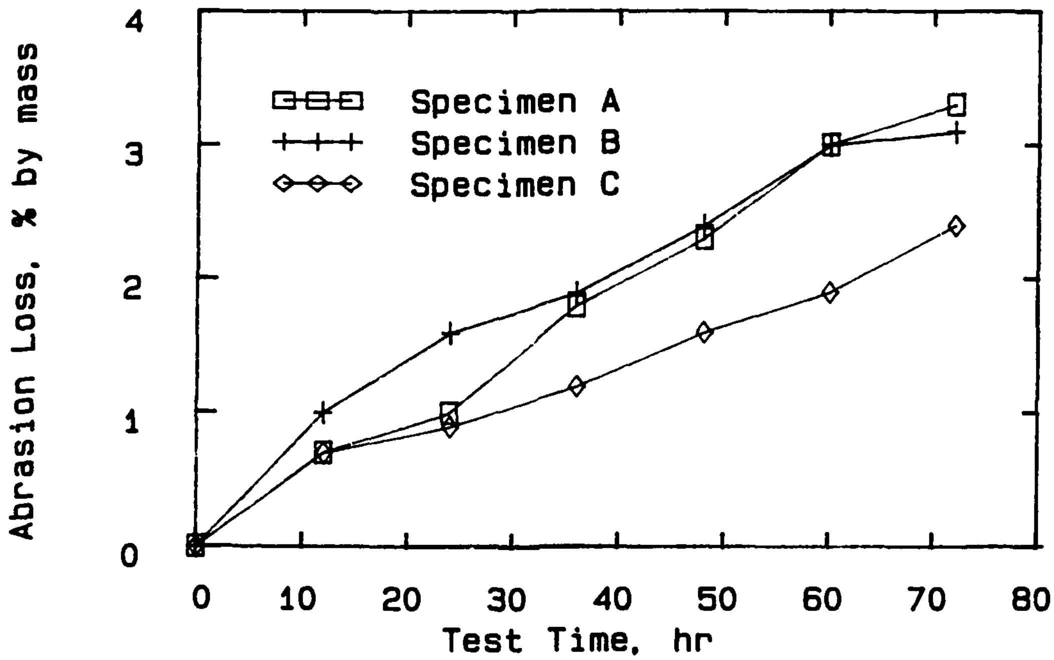

Figure 29. Abrasion-erosion test results

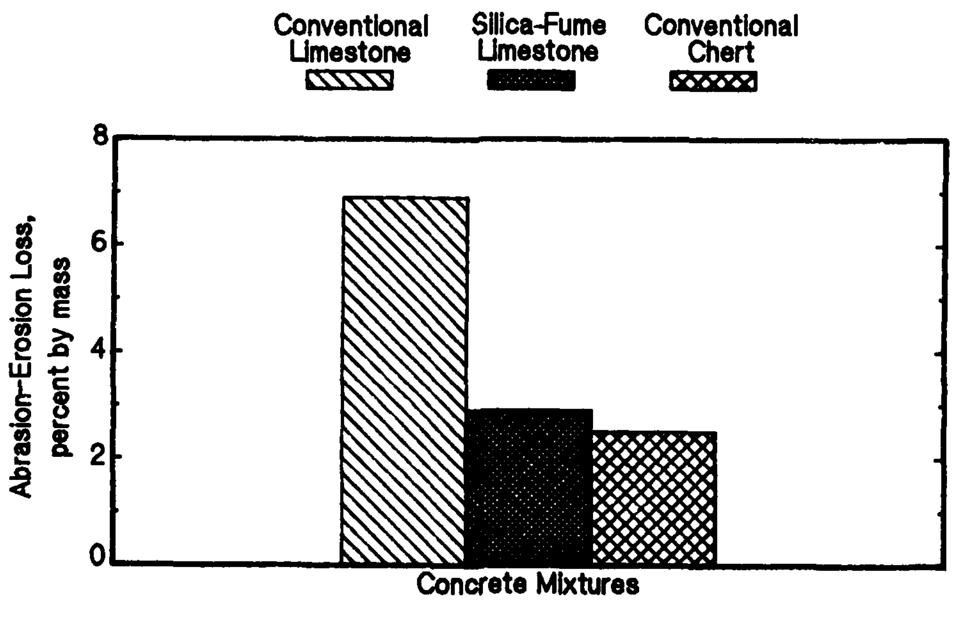

Figure 30. Abrasion-erosion resistance of limestone aggregate concrete with and without silica fume and HRWRA's compared to conventional chert aggregate concrete

PART IV: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

52. Concrete materials and mixture proportions similar to those used in the 1983 repair of the Kinzua Dam stilling basin were obtained for casting laboratory test specimens. The silica-fume content of this concrete mixture was approximately 18 percent by weight of cement. More recent work indicates that the optimum silica-fume content ranges from about 5 to 10 percent by weight of cement (ACI Committee 226 1987).

53. Adding silica fume and an HRWRA to a concrete mixture will greatly increase compressive strength, which, in turn, increases abrasion-erosion resistance. These very high-strength concretes appear to offer an economical solution to abrasion-erosion problems, particularly in those areas where locally available aggregate otherwise might not be acceptable.

54. The general approach to repair of mass concrete hydraulic structures has been to remove 1 to 2 ft of old concrete and replace it with new air-entrained concrete. One of the most persistent problems with this approach has been cracking in the replacement concrete (McDonald 1987). These early-age cracks are attributed to restrained contraction of the replacement concrete, the restraint being provided through bond to the existing stable concrete substrate. As the relatively thin layer of resurfacing concrete attempts to contract as a result of plastic and drying shrinkage, thermal gradients, and autogenous volume changes, tensile strains develop in the replacement concrete. When these strains exceed the ultimate tensile strain capacity of the replacement concrete, cracks develop. None of the material properties of silica-fume concrete reported herein, with the possible exception of autogenous volume change, indicates that this material should be significantly more susceptible to this type of cracking than conventional concrete. In fact, some material properties, particularly ultimate tensile strain capacity, would indicate that silica-fume concrete should have a reduced potential for cracking.

55. Silica-fume concrete requires no significant changes from normal transporting, placing, and consolidating practices. However, adding silica fume to a concrete mixture will reduce or eliminate bleeding which will affect

finishing procedures. This virtual elimination of bleeding causes rapid surface drying and, depending on atmospheric conditions, can cause plastic shrinkage cracking. Precautions against plastic shrinkage cracking are needed if the expected evaporation rate at the surface of conventional concrete approaches 0.2 psf (ACI Committee 305 1988). According to Holland (1987), this value is too high for silica-fume concrete in which cracking may occur if the estimated evaporation rate approaches 0.1 psf. Holland gives 11 suggestions for preventing plastic shrinkage cracking and 2 recommendations regarding finishing.

56. Proper curing of silica-fume concrete is essential, as it is for any concrete. However, improper curing is more harmful to silica-fume concrete than to conventional concrete (Holland 1987). To obtain the greatest benefit from silica fume, the concrete should be cured longer than conventional concrete. Starting immediately after finishing, silica-fume concrete should be kept moist with wet burlap for a minimum of 7 days.

57. Resurfacing of the lock walls at Lock and Dam No. 20, Mississippi River, resulted in significantly less cracking in the conventional replacement concrete than was previously experienced at other rehabilitation projects within the US Army Engineer District, Rock Island (Wickersham 1987). The reduced cracking was attributed to a combination of factors including lower cement content, larger maximum size coarse aggregate, lower placing and curing temperatures, smaller volumes of placement, and close attention to curing. Any variations in concrete materials, mixture proportions, and construction practices that will minimize shrinkage or reduce concrete temperature differentials should be considered in efforts to minimize cracking in concrete with and without silica fume. Guidance in these areas is given in EM 1110-2-2002 (Headquarters, US Army Corps of Engineers (HQUSACE) 1986) and ETL 1110-2-314 (HQUSACE 1988).

58. A general-purpose heat transfer and structural analysis finite element code was recently used to predict the response of concrete overlays placed on lock-wall surfaces (Norman, Campbell, and Garner 1988; Hammons, Garner, and Smith 1989). These analyses indicate that shrinkage is a predominant factor in overlay cracking. The analyses also demonstrate that an effective bond breaker at the interface between the replacement and existing concrete would eliminate cracking. WES recommendations to minimize shrinkage

and install a bond breaker were implemented by the US Army Engineer District, Pittsburgh, during 1989 in the rehabilitation of Dashields Locks, Ohio River. A recent examination of the project by Hugenberg* indicated that cracking of concrete placed during 1989 was significantly less than that of concrete placed during the previous construction season.

Recommendations

59. Silica fume offers potential for improving many properties of concrete. However, the very high compressive strength and resulting increase in abrasion-erosion resistance are particularly beneficial in repair of hydraulic structures. These concretes should be considered in repair of abrasion-erosion susceptible locations, particularly in those areas where locally available aggregate might not otherwise be acceptable.

60. The potential for cracking of restrained concrete overlays, with or without silica fume, should be recognized. Any variations in concrete materials, mixture proportions, and construction practices that will minimize shrinkage or reduce concrete temperature differentials should be considered. Where structural considerations permit, a bond breaker at the interface between the replacement and existing concrete is recommended.

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* Memorandum for Record, "Dashields Lock Rehabilitation, Site Visit," T. L. Hugenberg (1989), US Army Engineer Division, Ohio River, Cincinnati, OH.

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Table 1

Elastic Property Test Data

Age days		Compressive Strength, psi	Splitting Tensile Strength, psi	Modulus of Elasticity* psi x 10^6	Poisson's Ratio
1		5,910	565	4.0	0.20
		6,240	435	4.2	0.22
	Avg	6,080	500	4.1	0.21
3		8,290	675	4.9	0.22
		8,100	640	5.0	0.22
	Avg	8,200	660	5.0	0.22
7		10,670	725	5.8	0.25
		10,880	855	5.4	0.23
	Avg	10,780	790	5.6	0.24
28		14,270	875	5.9	0.23
		14,290	780	5.9	0.22
	Avg	14,280	830	5.9	0.22
90		14,400	995	6.4	0.23
		14,150	1,030	**	**
	Avg	14,280	1,015	6.4	0.23
365		14,890	820	6.8	0.24
		14,930	1,015	7.3	0.25
	Avg	14,910	920	7.0	0.24

* Secant modulus--50 percent of ultimate strength.
** Strain gage failure.

Table 2

Ultimate Strain Capacity Test Data

Beam No.	Test Age days	Total Load lb		Modulus of Rupture, psi	Tensile Strain Capacity,* millionths
1	1	17,300		600	120
6		14,800		515	128
			Avg	560	124
2	3	22,250		775	132
5		20,800		720	141
			Avg	750	136
3	7	29,000		1,010	155
4		29,275		1,015	160
			Avg	1,010	158
7	28	33,450		1,160	182
11		31,000		1,075	162
			Avg	1,120	172
8	90	35,250		1,225	**
9		34,400		1,195	223
			Avg	1,210	
10	365	32,000		1,110	168
12		36,000		1,250	200
			Avg	1,180	184

* Tensile strain at 90 percent of ultimate load.
** Strain meter failure.

Table 3

Adiabatic Temperature Rise Test Data

Time, hr	Temperature, °F
0	0
1.75	7.30
2.92	8.07
4.25	8.66
5.92	9.32
11.25	13.48
21.75	51.07
25.42	57.54
29.92	64.90
46.00	81.81
53.75	85.29
117.75	94.40
143.25	95.44
160.92	96.26
192.00	96.92
215.42	97.08

Table 4

Abrasion-Erosion Test Data

Elapsed Test Time hr	Specimen A Weight lb	Specimen A Percent Loss	Specimen B Weight lb	Specimen B Percent Loss	Specimen C Weight lb	Specimen C Percent Loss	Average Percent Loss
0	37.58	0.0	39.80	0.0	38.25	0.0	0.0
12	37.32	0.7	39.40	1.0	38.00	0.7	0.8
24	37.20	1.0	39.18	1.6	37.92	0.9	1.2
36	36.90	1.8	39.05	1.9	37.80	1.2	1.6
48	37.70	2.3	38.85	2.4	37.65	1.6	2.1
60	36.45	3.0	38.70	3.0	37.52	1.9	2.6
72	36.35	3.3	38.55	3.1	37.35	2.4	2.9